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JAN 79 E J BARENBERG, R E SMITH DOT-FH-11-8474

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LONGITUDINAL JOINT SYSTEMS IN SLIP-FORMED RIGID PAVEMENTS.

Volume I. – Literature Survey and Field Inspection.

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Ernest J./Barenberg
Roger E./Smith

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Ernest J. Barenberg Roger E. Smith

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16. Abstract

Most construction equipment for rigid pavements has been designed to place ribbons of concrete, called paving lanes, either 24 or 25 feet wide. Pavements of widths greater than 25 feet normally require the pavement be placed in parallel lanes with a construction joint between the lanes. To assure continuity of the pavement slabs across these lane-dividing joints, load transfer system are normally used, the most common of which for airport pavements is a keyway built into the pavement lane slabs. Construction of these keyway systems with the slip form pavers has caused some construction problems which result in slumping of the pavement edge and unsatisfactory load transfer across the joint. Furthermore, the high gear loads on modern aircraft have reportedly caused many of the keyway systems to fail thus causing serious maintenance problems on the pavements. This phase of the study was undertaken to evaluate the seriousness of this problem and to suggest possible solutions. Available literature on the design, construction and performance of various load transfer systems for rigid pavements were studied, and the potential for the various systems for use in the longitudinal joint with slip form pavers was evaluated. Visits were made to most FAA regional offices, a number of FHWA offices and to a number of airports to discuss this problem with knowledgeable field engineers. Results of these visits and the findings from the literature are presented in this report. **FAA-RD-79-4, Vol II, Analysis of Load Transfer Systems for Concrete Pavements and Vol III, Users Manual for Pavement Design are in preparation.**

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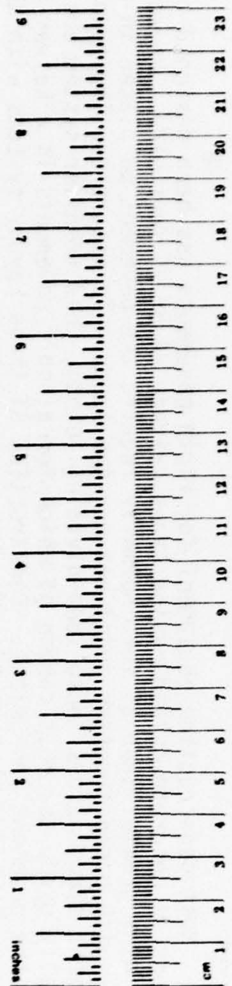
METRIC CONVERSION FACTORS

Approximate Conversions to Metric Measures

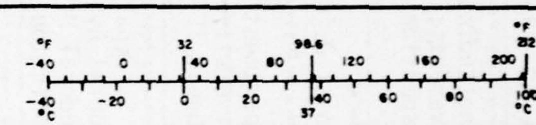
Symbol	When You Know	Multiply by	To Find	Symbol
		LENGTH		
in	inches	*2.5	centimeters	cm
ft	feet	30	centimeters	cm
yd	yards	0.9	meters	m
mi	miles	1.6	kilometers	km
		AREA		
in^2	square inches	6.5	square centimeters	cm^2
ft^2	square feet	0.09	square meters	m^2
yd^2	square yards	0.8	square meters	m^2
mi^2	square miles	2.6	square kilometers	km^2
	acres	0.4	hectares	ha
		MASS (weight)		
oz	ounces	28	grams	g
lb	pounds	0.45	kilograms	kg
	short tons (2000 lb)	0.9	tonnes	t
		VOLUME		
tsp	teaspoons	5	milliliters	ml
Tbsp	tablespoons	15	milliliters	ml
fl oz	fluid ounces	30	milliliters	ml
c	cups	0.24	liters	l
pt	pints	0.47	liters	l
qt	quarts	0.95	liters	l
gal	gallons	3.8	liters	l
ft^3	cubic feet	0.03	cubic meters	m^3
yd^3	cubic yards	0.76	cubic meters	m^3
		TEMPERATURE (exact)		
°F	Fahrenheit temperature	5/9 (after subtracting 32)	Celsius temperature	°C

*1 in = 2.54 (exactly). For other exact conversions and more detailed tables, see NBS Misc. Publ. 286, Units of Weights and Measures, Price $2.25, SD Catalog No. C13.10:286.



Approximate Conversions from Metric Measures

Symbol	When You Know	Multiply by	To Find	Symbol
		LENGTH		
mm	millimeters	0.04	inches	in
cm	centimeters	0.4	inches	in
m	meters	3.3	feet	ft
m	meters	1.1	yards	yd
km	kilometers	0.6	miles	mi
		AREA		
cm^2	square centimeters	0.16	square inches	in^2
m^2	square meters	1.2	square yards	yd^2
km^2	square kilometers	0.4	square miles	mi^2
ha	hectares (10,000 m^2)	2.5	acres	
		MASS (weight)		
g	grams	0.035	ounces	oz
kg	kilograms	2.2	pounds	lb
t	tonnes (1000 kg)	1.1	short tons	
		VOLUME		
ml	milliliters	0.03	fluid ounces	fl oz
l	liters	2.1	pints	pt
l	liters	1.06	quarts	qt
l	liters	0.26	gallons	gal
m^3	cubic meters	35	cubic feet	ft^3
m^3	cubic meters	1.3	cubic yards	yd^3
		TEMPERATURE (exact)		
°C	Celsius temperature	9/5 (then add 32)	Fahrenheit temperature	°F



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EXECUTIVE SUMMARY

Construction of longitudinal joints in slip formed concrete pavements has caused special problems for both the contractor and the pavement maintenance engineer. For the contractor these problems are related to construction of the slip formed edge with load transfer devices. Maintenance problems are caused because the most popular method of load transfer in longitudinal joints, namely the keyway systems, frequently fail under today's heavy aircraft loading.

Reports from the field indicate a significant incidence of failure in keyed joint systems. There is some evidence that tying the keyed joints will reduce these failures, but tying keyed joints with slip form pavers have caused serious construction problems. Thus alternate solutions are needed to provide adequate load transfer in longitudinal joints constructed with slip formed pavers.

Objectives

The objective of this total study was to determine the cost effectiveness of various load transfer systems across longitudinal joints in slip formed concrete pavements. Ultimately, the study will provide a rating system for various load transfer systems in terms of cost effectiveness of some of the more promising systems.

Scope

This study is broken into several phases. This report covers the background studies and includes the findings from the literature survey and from field inspections of existing pavements with keyed load transfer systems and of construction of pavements with slip form pavers. Some alternate joint systems developed as a result of the literature survey and discussions with

field personnel are presented herein, but no analysis of the proposed joints are included in this report. Volumes II and III will include the methodology and results of the analysis of the alternate joint systems.

Information Gathering

Information for this phase of the study was collected from the published literature and from visits to a number of airports with jointed concrete pavements. The survey of published literature covered the entire range of joints and load transfer systems rather than just the longitudinal joints. Likewise, in the field investigation, all joint types were observed and evaluated with the understanding that some of the load transfer devices used in other types of joints may also be cost effective as load transfer systems for slip formed longitudinal joints.

Contact was made with all of the FAA Regional offices to assist in selection of the airports to be visited in the field survey. Contact was also made with a number of FHWA field offices to determine how the information being collected could also be applied to highway pavements. Visits were made to 14 airports located in 7 of the 9 FAA regions. Only the New England and Western Regions of FAA had no airport pavements included in the field survey.

At each location visited the project staff met with engineers from the FAA regional offices, airport engineers and consultants to discuss the particular pavement joint problems, their seriousness and possible solutions. Inspection tours were also made of the pavements at the airports visited so the project staff could see firsthand how the load transfer systems were performing and the specific problems thereto.

Based on these field surveys and the findings from the literature, it

was concluded that keyways, while very effective as load transfer systems before failure, are also the source of many problems in the construction of slip formed rigid pavements. Specifically it was determined that it requires a high level of quality control on the part of the contractor to properly construct the keyways in slip formed pavements, and once constructed there is a tendency for the keyways to shear off, especially the female side, thus causing severe maintenance problems. The failure problem is much more serious in thin pavements loaded to near their capacity than in the thicker pavements.

Through the conversations with FAA field engineers, airport engineers, contractors, and consultants plus results from the literature, some alternate load transfer systems for slip formed rigid pavements were developed. Descriptions of some of the more promising of these are included in this report. No analysis is given in this report as to the relative cost effectiveness of the various joint-load transfer systems as this will be included in Volume II of the reports from this study. Some of the alternate joint systems have been installed at selected airports and these installations can provide a basis for making comparative performance analyses of the alternate systems.

I. Introduction

Problem

Slip-forming of Portland cement concrete pavements is authorized by most agencies for construction of airport and highway pavements. In general, the quality of construction using slip-formed pavements is high and gives satisfactory performance. There is a problem associated with the slip-forming method of construction in the construction of effective and economical load transfer devices along the longitudinal joints. In general, load transfer devices presently used in slip-formed construction are no different from the devices used in conventional formed construction. But, when used in slip-formed construction, construction of these devices can cause problems in addition to those encountered in conventional construction. Keyways, especially for thick pavements, are subjected to edge slumping if the concrete is too wet, honeycombing results if the concrete is too dry. For the thiner pavement sections, i.e., less than 12 inches in thickness, there is a serious problem of shearing of the keyway sections under heavy aircraft gear loads. This also occurs in some of the thicker sections, but not as frequently. Dowels installed in plastic concrete during slip-forming can cause bulging at the surface, misalignment, and edge slumping. As a result, dowels have been installed, in some cases, in hardened concrete by drilling holes in the pavement edges. The drilling and installation can damage the concrete, and the dowels require grouting which results in added steps and cost of construction. Thickened edges, especially on stabilized bases, are difficult to construct. Improvements of the present load transfer devices, and development of new innovative devices are urgently needed to increase construction efficiency and effectiveness of slip-formed pavements.

Objective

The objective is to determine the cost effectiveness of load transfer systems across longitudinal joints of slip-formed pavements for civil airports. The end product desired is a numerical rating in terms of the cost effectiveness and performance of the most promising of the load transfer systems.

Scope

This effort will consist of : (1) an evaluation of load transfer devices installed in slip-formed construction projects, (2) investigation of concepts for new load transfer systems, and (3) development of criteria and analysis of load transfer systems. The analysis is expected to provide results in terms of joint design criteria and the cost effectiveness of each load transfer system. It is anticipated that the most cost effective load transfer systems developed from this program will be evaluated in future ADAP construction. Field installation and evaluation are not a part of this study.

Approach

An evaluation of past and current practices for constructing load transfer devices with the slip-form method of construction was made. This was done by making a thorough review of the literature on this subject, and by field inspection trips to airports under construction or where problems had previously occurred. Interviews were conducted with airport engineers, consultants to airport authorities and with contractors, to determine construction problems and solutions, and the performance of the various tvoes of load transfer systems.

New and innovative load transfer systems were proposed and discussed with engineers during the field trips. Some of these are now under

analytical investigation and so will not be reported on until the final report is prepared.

One of the more challenging tasks in this study is that of establishing criteria for evaluating the various load transfer systems. Some preliminary work has been done on this and the approach being taken is discussed later in this report. Final conclusions on this subject will not be presented, however, until the final report.

This interim report primarily covers the findings from the literature survey and results from the field visits. Limited information is presented on some of the new load transfer systems, on criteria for evaluating joint efficiency, and on the investigator's approach for establishing the most effective longitudinal joint systems. These later items will be presented in detail in the final report to the contractor.

II. Field Visitations

Field visits were made to a number of airports to examine first hand the types of problems encountered with the various load transfer devices. Two types of problems were evaluated during the field visits: performance of the various load transfer devices, and construction problems associated with their installation.

Sites for the visitations were selected in the following manner. First, all of the regional offices of FAA were contacted, both in writing and by phone, and the problems being addressed were discussed with them. Based on the comments and recommendations of the FAA Regional Pavement Engineer, a number of airfields were tentatively selected for visits. The engineer in charge of pavements at these airports were then contacted and the problem discussed with them. After this interview, a decision was made as to the value of the proposed visits and whether to proceed with the scheduled visits. In several instances additional visits were then scheduled if such visits appeared to have potential for providing useful information, and if the additional visits could be arranged at only a nominal additional cost. For example, the visit to the Kansas City Municipal Airport was made as a part of the scheduled trip to the Wichita airport with the only additional cost being one additional nights lodging and one additional day of per diem. It is important to note that some of the most valuable data and information were collected from airports which were scheduled as secondary visits.

Whenever a visit was scheduled to a particular airport a check was made to determine if FHWA Regional or District offices could be reached without significant additional cost or time. If so, the particular offices were contacted for possible information on the problem under study.

Visits were made to the following airports and offices as a part of this study.

FAA Region	Airports Visited
ANE (New England)	None
AEA (Eastern)	Pittsburgh, PA
ASO (South)	Jacksonville, FL
AGL (Great Lakes)	Chicago-Midway, IL
	Indianapolis, IN
	Detroit, MI
	St. Louis, MO
ACE (Central	Wichita, IS
	Kansas City, MO
ASW (Southwest)	Shreveport, LA
	Baton Rouge, LA
	Houston, TX
ARM (Rocky Mountain)	Denver, CO
ANW (Northwest)	Seattle, WA
	Spokane, WA
AWE (Western)	None

A summary of what was observed and some general comments on the findings are presented as trip reports in the appendix of this report. Literally scores of photographs were taken, not only of the joints, but of all types of distress and construction procedures during these trips. Only a few selected photographs which illustrate points germane to the objectives of this study are included in the trip reports. The rest of the photographs are on file as slides in the contractor's fiels. These will be produced as requested by the project monitor.

Since the results from each field trip are included in the appendix these will not be discussed here individually, but the findings will be incorporated into the results from the literature search. Where appropriate reference will be made to the specific visit where the distress or construction problem was observed.

III. Slip-Form Paving Experience

The slip-form paving method is the principal method used in construction of rigid airport pavements. This procedure creates problems in edge slumping, particularly on thick slabs, and in construction of adequate load transfer mechanisms.

Investigation of edge slumping by F. Parker, Jr. (26) revealed that edge slump is roughly proportional to the thickness of the slab. In addition, the transverse distance affected by slump increases as the slab thickness increases. During construction, contractors have tried to touch up the edge by manual finishing. Parker and Brandley (3) recommend that manual finishing be kept to a minimum as it tends to increase the variations. Parker concluded, that riding characteristics of the surface are not adversely affected when edge variations exceed 1/4 inches (6.35 mm) in only a few locations.

A number of methods have been used in attempts to reduce edge slump. The edge geometry and the materials used at the edge have been altered. The edge has also been buttressed with a wedge of gravel to hold it in place. However, these attempts sometimes cause overcompensation or adversely affect durability (26).

Load transfer systems most commonly used with the slip-form paver in construction of longitudinal joints is the keyed joint. Keyed joints have been constructed both with and without tiebars, and have been constructed with either the male or female side of the joint cast first.

The major problem associated with construction of this type of joint has been the sloughing of the concrete around the joint. If the concrete is placed too wet the sloughing becomes critical, and if placed too dry there is a tendency toward honeycombing around the keyways. Parker (26) reports

that inadequate consolidation of the concrete around the keyway forming device is a primary cause for sloughing. Inadequate consolidation will also result in reduced quality of concrete and the ultimate deterioration of the joints as observed at Denver and Indianapolis Airports.

One method used to reduce sloughing during formation of the keyways is to use metal keyway liners in the formation of the female side of the keyway as illustrated in Figures 1 and 2. Two types of these liners were used in construction of the concrete overlay at the Wichita Airport, one a preformed metal device supported on chairs at a predetermined level above the subbase, and the other a liner formed from a flat metal strip by a series of rollers as it is fed into the paver during placement. Both types of liners were intended to be left in place. It was found, however, that the 27 gage metal used at Wichita in the liners formed by the paver during placement may have been too light, as construction activity caused the liner to be pulled away from the top lip of the keyway in some locations. When this happened the metal liner had to be removed prior to placement of the adjacent lanes of concrete, otherwise a gap would exist between the elemnts of the two sides of the keyway, thus eliminating the load transfer efficiency of the joints. The 24 gage metal used in the preformed liners at both Wichita and SEATAC Airports did not have this pull away problem, and reportedly worked very well in the formation of the joints. Placement of bent, deformed tiebars through preformed holes in the metal liners reportedly did not cause any particular problems during construction.

Formation of the male side of the keyways first has led to considerable construction problems. This is especially true when tie bars or dowels were inserted into the keyway while the concrete is still plastic. Such practice

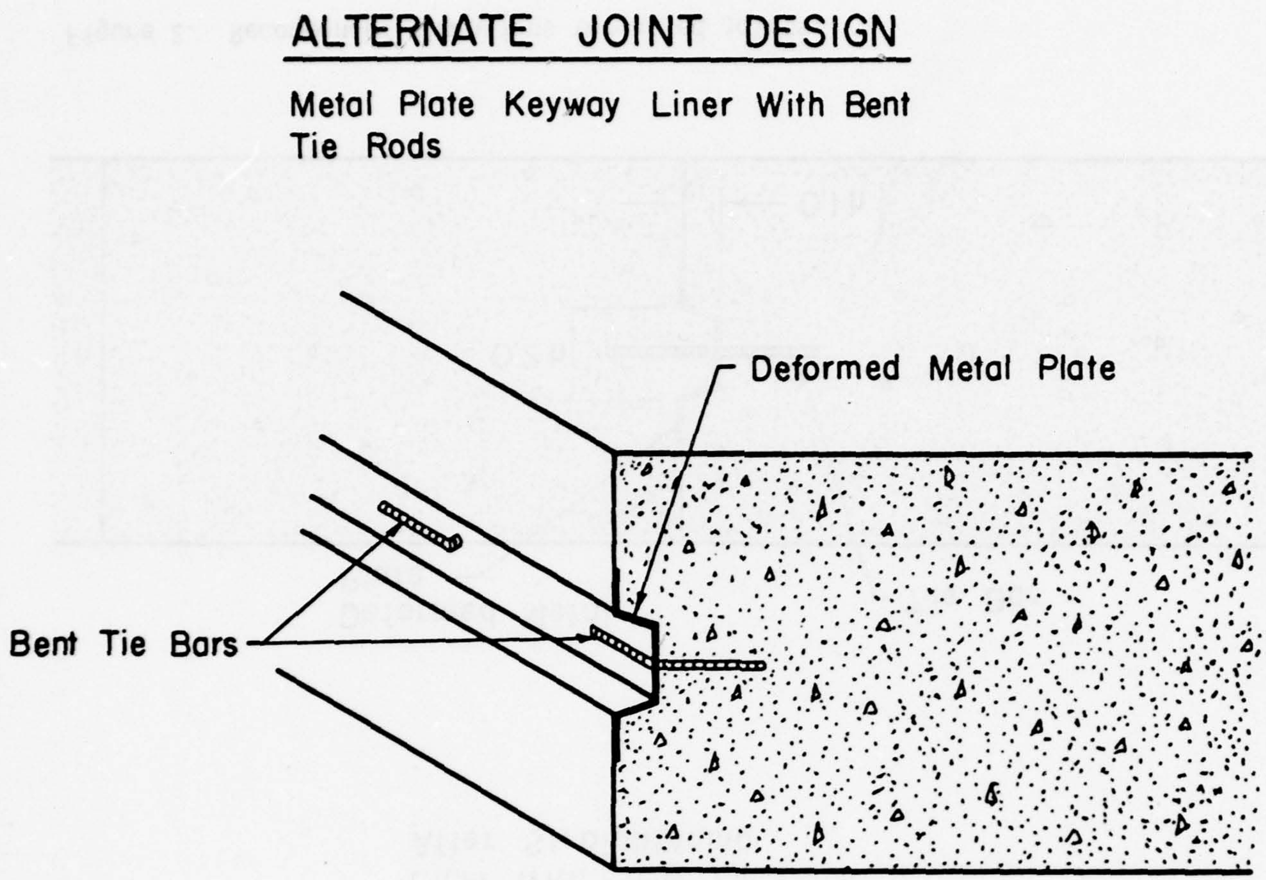


Figure 1. Current Procedure for installation of tie bars in keyed pavements.

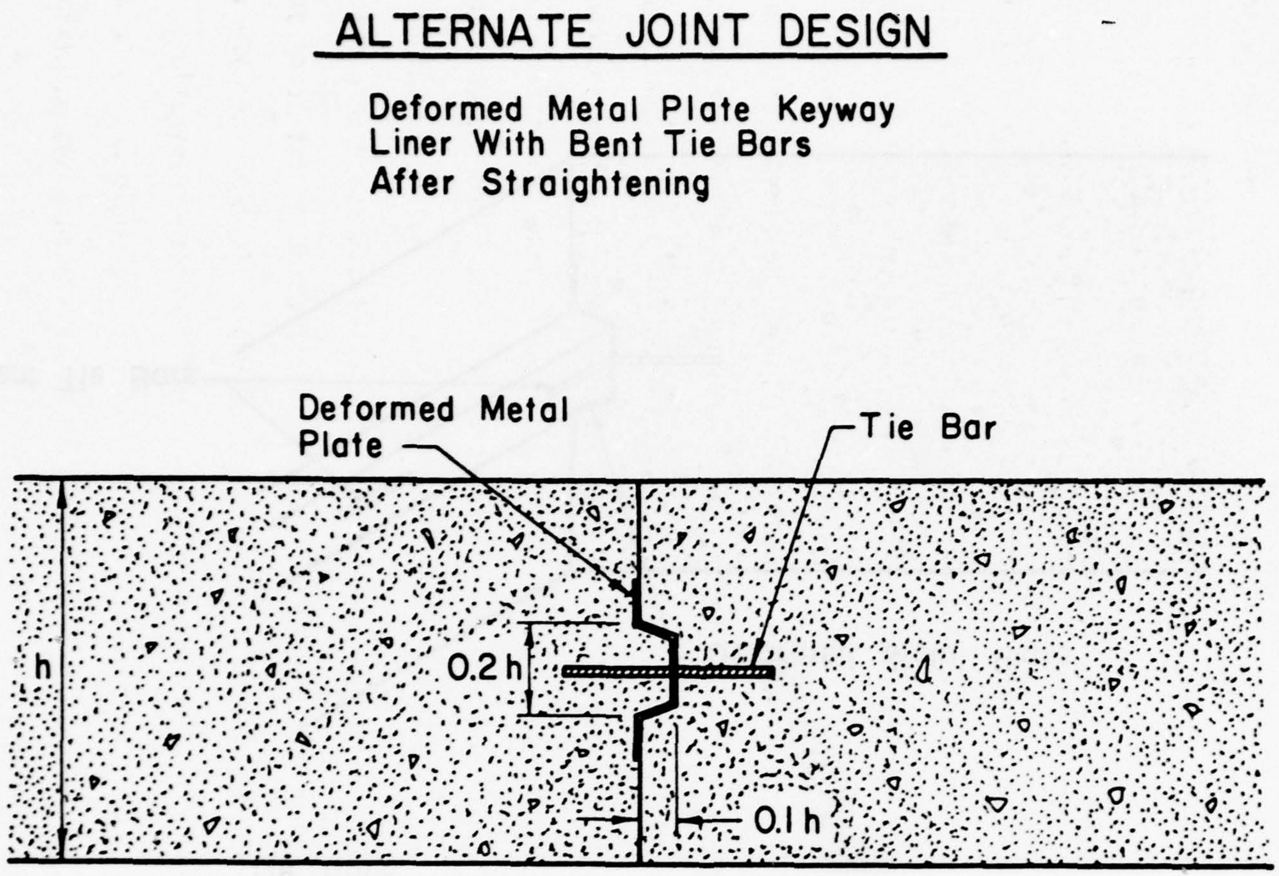


Figure 2. Recommended dimensions for keyed joints.

inevitably causes sloughing, of the keyway and often loss of the entire keyway in an area. The trip reports from the Detroit, Pittsburgh and Shreveport Airports discuss the various aspects of this problem. In construction at the Pittsburgh and Shreveport Airports, for example, the problems were so severe that use of the keyed joints were abandoned completely. In construction at the Detroit Airport, use of keyed joints was continued, but with the provision that whenever significant keyway sloughing occurred the contractor would, at no cost, drill and grout in dowels to serve in lieu of the lost segments of the keyways.

While most resident engineers and contractors indicated a strong dislike for the construction of keyed joints in slip-formed concrete pavements, many agreed it could be accomplished. The 27 inch thick pavement on the Houston was apparently placed without significant problems even though the male portion of the keyway was formed first and without any type of support. A probable reason for this success was that the contractor on that job was well-known for its quality of work. A similar situation was observed in the construction at the Wichita Airport where the contractor kept careful control of both the concrete plane operations and the placement operations. This quality control effort again paid off with a minimum of problems with respect to the construction of the keyed joints. Conversely, for the construction of other airports visited, the material supplier and the paving contractor were from different organizations. This loss in control was reported by the responsible engineers to be a major factor in the paving contractor's inability to form and maintain a satisfactory keyed joint.

Doweled longitudinal joints were given a much more favorable rating by the paving contractors and engineers visited. Most engineers and contractors

visited indicated little or no problem in installing the dowels in the plastic concrete where there were no keyways to contend with. In some instances large dowels (1 1/4 or greater) reportedly caused some sloughing along the edges but this was not considered to be serious. Dowels less than one inch in diameter can apparently be placed in the plastic concrete 12 inches or more in thickness with almost no edge slumping. Installation of deformed tie bars into the plastic concrete cause about the same level of problems as the dowel bars. However, since the tie bar diameter is usually much less than that of the dowels for the same pavement thickness, the problems reported are also less. Insertion of dowels into plastic concrete is often accompanied by poor placement control and poor dowel alignment.

Drilling and grouting dowels into hardened concrete reportedly poses no particular construction problems, but it is expensive. Several contractors and engineers with whom this problem was discussed indicated the cost could probably be reduced significantly if the job were planned so that gang type drills could be developed. From a strictly operational point of view the installation of dowels by drilling and grouting appears to be the most favored by the contractor. Some recent contacts with consultants and contractors indicate gang drills have been employed at several airports. Since the authors have no hard data on these installations at this time, the specifics of this approach will be discussed in a later report.

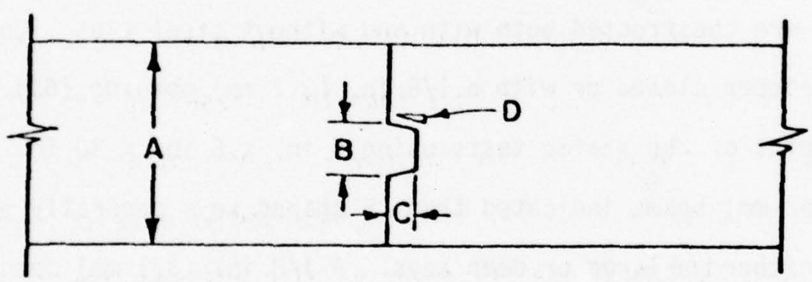
IV. Performance of Current Load Transfer Systems

A joint in concrete pavements are a point of structural weakness which limits the load carrying capacity of the entire pavement system (68). This fact has been the subject of much investigation since the first concrete pavement was constructed in Inverness, Scotland, in 1865 (67). Specifications for primarily transverse joints were first adopted by the American Concrete Institute at its tenth annual convention in 1914. Dowels were first used for load transfer systems in 1917 (65, 67), and their use spread widely and quickly. During the decade of the 1920's, the use of longitudinal joints to divide pavements into smaller slabs (approximately 10 feet [3.05 m]) was adopted. Tongue and groove load transfer systems for transverse joints were patented between 1923 and 1929 (67). Both doweled and keyed longitudinal joints were constructed and tested in the 1930's by the Bureau of Public Roads (67). These two systems have been studied and refined since, and still remain the primary load transfer systems used in longitudinal construction joints today.

A. Keyed Joint Performance

The performance of keyed construction joints has been studied by several investigators. Among the most comprehensive studies are those by the U. S. Army, Corps of Engineers, Waterways Experiment Station (WES), Vicksburg, Mississippi. Rice (32) reported the results of tests conducted on several thicknesses (8, 10, 12, 14 in.) (203, 254, 304.8, 355.6 mm) of keyed PCC pavement subjected to repeated heavy simulated aircraft loadings with the key formed in accordance with U. S. Army, Corps of Engineers (CE) specifications (See Figure 3). In these tests, the center of the key was located at the center of the slab depth. The pavements were constructed on a subgrade of heavy clay compacted to provide a foundation strength of about 4 CBR or a modulus of about 100 PCI (27.1 N/cm^3). The keyway systems failed in all four sections after less than 1000 passes of simulated aircraft load, with the key failures nearly as prevalent as the keyway failures. Based on these results, Rice recommended that keyed construction joints be avoided in PCC pavements for heavy aircraft when constructed on low strength foundations.

Grau (13) also reported on a series of tests conducted at WES to evaluate load transfer systems in PCC pavements. An 8 inch (203 mm) keyed PCC pavement was constructed on a medium strength base of clayey, gravelly sand 24 inches (609.6 mm) thick compacted to 103 percent CE55(CT-611) over a heavy clay, and a 10 inch (254 mm) keyed PCC pavement constructed on a 6 inch (152.4 mm) course clayey gravelly sand stabilized with 6 percent cement again over heavy clay. The keyways were constructed with the dimensions as recommended by Rice (13). The keyed joint in the 8 inch pavement on the medium strength base failed nearly simultaneously with the slab itself. The keyed joint in the 10 inch pavement on the cement stabilized



A - Slab thickness

B - Vertical depth of key = 0.2A

C - Horizontal depth of key= 0.1A

D - Slope of bevel of key= 1:4

Figure 3. Corps of Engineers recommended dimensions for keyed joints.

base did not fail. Grau (13) concluded therefore, that keyed construction joints in PCC pavements would probably perform adequately on high strength or stabilized soil foundations.

It should be noted that in both series of tests reported above, the PCC pavement was placed using conventional fixed forms rather than with a slip-form paver.

The Portland Cement Association (PCA) performed a series of tests on load transfer systems. Concrete beams containing keyed joints were tested statically and repetitively. Keys with three different dimensions were tested; U. S. Army Corps of Engineers (CE) dimensions, as shown in Figure 3, a key with double height (large key), and a key with double depth (deep key). Keys were constructed both with and without steel ties. Joints were tested either closed or with a 1/8 in. (3.2 mm) opening (69).

The results of the static tests using 6 in. x 6 in. x 30 in. (152 mm x 152 mm x 762 mm) beams indicated that CE shaped keys generally performed better than either the large or deep keys. A 1/8 in. (3.1 mm) opening was detrimental to the load transfer capacity of the system. Tied keys performed better than untied, and keys tied with deformed tie bars performed better than those with plain or elastic tie bars (69).

Beams 12 inches (304.8 mm) wide by 48 inches (1.22 m) long and 18 inches (457.4 mm) thick were also tested under static loads. These tests results indicated that untied keyway systems performed poorly; however, keyway systems tied with No. 4 deformed bars performed as well or better than beams 18 inches (457.4 mm) thick and no load transfer systems (69).

The results from the repetitive tests showed that untied keys using

the Corps of Engineers dimensions performed poorly while the tied keys with the same dimensions were ranked in the middle of each group of tests. These results are summarized in Table 1 (69).

The researchers made the following general conclusions from the results obtained: performance at the keyways is closely related to the tie steel; sharp fillets should be avoided on keys; and a cement-treated subbase significantly improves the joint performance (69).

Field performance of keyed joints appears to substantiate the conclusions reached by the researchers. Observations of key failures during the field visits were generally in pavements which had been loaded in excess of the initial design loads and the keys had not been tied. One possible difference in the field performance from that reported by the researchers is that keyway failures were observed in pavements with keyed joints built on strong bases and subbases, and with relatively light traffic. The pavements at Midway in Chicago, for example, show significant keyway distress along the centerline joint even though the pavements were overlays constructed on the existing pavement system. Other examples, reported to the researchers which cannot be documented here because of possible litigation, suggest that keyway failures can occur in concrete overlays before any aircraft traffic is placed on the pavements. Construction traffic may, however, have had an influence on these failures.

All of the significant keyway failure examples observed in the field inspections were seen on pavements with slab thicknesses of 12 inches or less but carrying major airline traffic (727's or heavier). Among the most severe keyway failure distress was that observed in the pavements at Wier Cook Airport, Indianapolis, Indiana. The slab thickness on these pavements was reportedly 11 inches in the areas of severe joint distress. In the Jacksonville, Florida Airport, the center 6600 foot

Table 1. Specimens Loaded Repetitively.

Test no.	Joint Type	Joint Ties	Slab Depth, in.*	Slab f_c, pci	Support k, pci	Initial load, kips	"Leave" deflection Initial	"Leave" deflection Final	Duration, cycles (million)	Rank within group***
1	AI**	0	6/0	4300	80	7.2	0.045	0.022	0.63	8
2	AI	2-No. 2	6/0	4150	70	7.5	0.048	0.035	2.14	3
3	CE key	0	6/0	4080	65	7.7	0.045	0.025	0.74	7
4	CE key	2-No. 2	6/0	4060	72	8.0	0.047	0.034	1.63	4
5	Smooth	2-No. 2	6/0	4220	84	6.0	0.046	0.032	1.51	6
6	Dowel	2-3/4 in.	6/0	4350	75	6.3	0.047	0.034	2.66	5
7	Elastic	2-No. 2	6/0	4070	68	6.8	0.049	0.038	3.41	1
8	Round key	2-No. 2	6/0	4260	72	7.1	0.049	0.036	3.27	2
9	AI	0	6/3	3970	220/70	7.9	0.047	0.031	1.16	3
10	Smooth	2-No. 2	6/3	3930	190/65	8.1	0.048	0.035	2.60	2
11	Dowel	2-3/4 in.	6/3	4000	250/75	8.5	0.049	0.037	3.21	1
12	AI	0	8/0	3860	75	7.6	0.046	0.026	1.10	8
13	AI	2-No. 2	8/0	4100	70	8.3	0.049	0.039	3.85	1
14	CE key	0	8/0	4090	72	7.8	0.046	0.028	1.26	7
15	CE key	2-No. 2	8/0	4250	80	8.4	0.049	0.035	2.18	4
16	Smooth	2-No. 2	8/0	4140	85	7.8	0.047	0.032	1.62	6
17	Dowel	2-3/4 in.	8/0	3930	75	8.0	0.048	0.035	2.06	5
18	Elastic	2-No. 2	8/0	3900	78	8.1	0.048	0.037	3.66	2
19	Round key	2-No. 2	8/0	4270	80	8.4	0.049	0.036	2.95	3
20	AI	0	6/6	4030	350/75	8.6	0.047	0.029	1.83	3
21	Smooth	0	6/6	4160	320/68	7.8	0.046	0.025	0.96	5
22	Smooth	2-No. 2	6/6	3930	380/80	8.0	0.047	0.027	1.68	4
23	Dowel	1-1 in.	6/6	3890	360/75	8.5	0.048	0.039	2.36	2
24	AI	2-No. 2	6/6	4110	340/70	8.8	0.049	0.041	4.44	1

*Depth denotes both the slab thickness and the cement-treated subbase thickness.

**AI = aggregate interlock.

***1 = best performance; 8 = poorest.

Ref. pg. 17 (69).

segment of the runway with the 11 inch thick pavements reportedly had significant keyway failures, whereas the 600 feet on both ends of the same runway which had 13 inch thick pavements, showed no evidence of keyway distress. The authors did not see the distress in the central position of the runway as it had been removed prior to the visit, but the 500 feet on both ends were left intact and showed no evidence of keyway distress.

As indicated in the trip reports, the keyways in Denver and Indianapolis Airports showed significant distress probably magnified by the deterioration of the concrete and by the level of stresses in the keyway area.

Conversely, pavements with keyways in thick pavements showed very little evidence of distress. SEATAC Airport (Seattle-Tacoma, Washington) for example, with thickened edge pavements up to 18 inches showed no evidence of keyway distress. Discussions with airport engineers at other airports having slabs with thicknesses of greater than 12 inches all indicate no particular distress in the keyed longitudinal joints.

IV-B. Doweled Joint Performance

Because of their extensive use in highway pavement, there have been many more testing programs on doweled joints than on the keyed joints. The U. S. Army Engineers at WES (13, 32) and ORDL (78, 79), the U. S. Navy (90) Bureau of Public Roads (67, 68, 70, 86) and the Portland Cement Association (PCA) (69) have all conducted extensive research on doweled joints for concrete pavements. In addition many papers and reports have been written on the subject, some from the research indicated above and some by private institutions and industry (28, 29, 32, 34, 38, 54, 55, 65, 67, 69, 70, 75, 78, 79, 84, 85, 86, 87, 88, 89, 90, 95, 96, 97).

All of the above reports have been received but not all are discussed in detail in this report. Instead, those reports and conclusions which, in the opinion of the project staff, are most pertinent to this project are presented here as background material. Some of the results given in other reports will be incorporated with the final reports on the analysis and optimization of longitudinal joints in concrete pavements placed with slip-form pavers.

One set of tests directly applicable to this study is that conducted by the U. S. Army Corps of Engineers WES. Dowels were installed as load transfer systems in three sections of a test pavement. One was a 10 inch (254 mm) PCC pavement placed over a 4 inch (101.6 mm) thick sand filler course over a low strength clay subgrade. The second pavement section was an 8 inch (203.2 mm) thick PCC pavement constructed on a medium strength base of clayey, gravelly sand 24 in. (609.2 mm) thick over a subgrade of low-strength, clay (See similar section for keyed joint). The third section was a 10 inch (254 mm) thick PCC pavement constructed on a 6 inch (152.4 mm) thick cement stabilized clayey gravelly sand over

a low-strength, clay subgrade. A similar section was tested with a keyed joint. The test results indicated the doweled longitudinal construction joints performed satisfactorily in all three sections (13). In contrast to these results, several of the identical pavement sections with keyed joints failed during the test (Chapter 3).

The PCA (Portland Cement Association) also tested dowels as load transfer systems in a contraction type joint. The dowel load transfer systems allowed horizontal movement and were tested with openings of 1/8 inch (3.2 mm) to 3/4 inch (19.1 mm) between the concrete faces. Solid round dowels 1/2 inch (12.7 mm) and 3/4 inch (19.1 mm) in diameter were tested statically in beams 6 inches (152.4 mm) by 6 Inches (152.4 mm) by 30 inches (762 mm). The dowel system did not perform as well as the tied keyway system; but, this was attributed to the joint openings used with the doweled joints. Dowel systems with 1/4 inch (6.35 mm) and 1/2 inch (12.7 mm) openings were rated better than the keyed system with a 1/8 inch (3.2 mm) opening (69).

Static tests using beams 12 inches (304.8 mm) wide, 6 inches (152.5 mm) thick and 48 inches (1.22 m) long, were tested with 3/4 inch (19.1 mm) solid steel dowels. Further tests were made with one inch (25.4 mm) dowels in beams 8 inches (203.2 mm) thick, and 1 1/4 inch (31.75 mm) dowels in beams 10 inches (254 mm) thick. The dowels performed well in all cases; but it was noted that joints with small (1/8 inch [3.2 mm]) openings performed better than those with large (1/2 inch [12.7 mm]) openings (69).

Dowels 3/4 inch (19.05 mm) in diameter were also studied under a repeated load in a 6 inch (152.4 mm) thick slab with a 1/4 inch (6.4 mm) opening. This combination of dowels and joint opening performed

comparably to the tied keyway system on a subgrade with a low "k" value. When the doweled system was tested on a cement-treated subgrade, it performed better than any of the other 6 inch (152.4 mm) thick systems tested (69) (See Table 1).

The authors of the PCA report concluded the conventional dowels provide load transfer as effective as any load transfer device tested (69). Teller and Cashell reported on the development of test equipment and test procedure for repetitive loading of load transfer systems. They tested a series of concrete slabs 48 in. (1.22 mm) wide with four dowels spaced 12 in. (305 mm) center to center. Slabs 6 in. (152.4 mm), 8 in. (203 mm), and 10 in. (254 mm) thick were used. Deflection of the slabs, average shear in the load-transfer system and shear value of single dowels were measured. In addition, the two outer dowels were cut, and the average shear and deflection values were again measured with only two dowels active and compared with the previous results with 4 dowels active (86). From these list results Teller and Cashell made the following conclusions (86):

1. "A definite exponential relation exists between dowel diameter and load-transfer capacity, other conditions being constant.

2. "A relation exists between slab depth and the dowel diameter required to transfer a given percentage of the applied load. This relation may be expressed as an approximate rule for minimum dowel size, as follows: For round steel dowels at a 12-inch spacing in joint openings of ¼-inch width or less, the dowel diameter in eighths of an inch should equal the slab depth in inches.

3. "The length of dowel embedment necessary to develop maximum load transfer is not a constant function of dowel diameter as has sometimes been assumed. With a 3/4-inch dowel diameter, maximum load transfer requires an embedded length of about 8-dowel diameters. With larger dowels, such as the 1-inch and 1 ¼-inch diameters now in common use, full-load transfer is obtained with a length of embedment of about 6 diameters, both initially and after many hundreds of thousands of cycles of repetitive

loading. The use of shorter dowels in these larger diameters would, in many cases, result in an appreciable savings in the amount of steel required for dowels.

4. "For a given dowel diameter and condition of loading, decreasing the width of joint opening decreases the bending stress in the dowel. It decreases also the dowel deflection and hence increases the percentage of load transferred, both initially and after extended repetitive loading. It is evident that a given load-transfer system may be expected to give a much better structural performance in a contraction joint than in an expansion joint of 3/4-inch width or greater.

5. "The condition of dowel looseness has an important effect on the structural performance of the dowel, since it can function at full efficiency only after this looseness is taken up by load deflection. This is true for both initial looseness and that which develops during the course of repetitive loading. Tests which do not include repetitive loading and complete stress reversal provide no information on this important condition and no measure of its effects.

6. "The application of extended repetitive loading decreases the initial ability of a given system to transfer load. Under equal conditions, the amount of this loss varies considerably as dowel diameter, length of dowel embedment, and width of joint opening are varied."[1]

These conclusions by Teller and Cashell especially with respect to the effect of repetitive loading are highly significant with respect to the direction which should be taken on future studies involving doweled load transfer systems. These results along with those reported by Ball and Childs of PCA (69) indicate that unless the horizontal movement is really needed deformed tie bars-dowels would be a much more effective load transfer scheme.

The U. S. Naval Civil Engineering Research and Evaluation Laboratory, Port Hueneme, California conducted a study on the load transfer characteristics of dowels for airfield pavement expansion joints. The slabs used in this study were 10 in. (254 mm) thick, 15 ft. (4.572 m) wide, and 25 ft. (7.62 m) long joined by a 3/4 in. (19 mm) wide expansion joint through which 1/8 in. (2.86 mm) diameter dowels were placed on 12 in. (304.8 mm) centers. Each dowel was instrumented with strain gages. Moment, shear,

and pressure on each dowel as well as slab deflections were measured. The researchers concluded that joints should not be designed completely on the basis of the theoretical value of a single dowel because the action of each dowel is directly related to the performance of other dowels in the joint. Their results indicated the five center dowels carried about 77 percent of the transferred load when the load was placed over the center dowel. Furthermore, these five dowels transferred from 32.5 to 38.0 percent of the total applied load to the adjacent slab (90).

No known doweled, longitudinal joints were inspected during the field inspections. Use of doweled construction joints were discussed with a number of pavement engineers, and contractors and all seemed to feel that from a construction point of view the doweled, longitudinal joint would be much easier to construct than the keyed joints discussed earlier. The basic question then is to determine the field performance of doweled joints in airfield pavements. Knowledge of the field performance is especially critical in view of the results reported by Teller and Cashell (86) concerning the loss in efficiency of these joints under repeated loads.

The U. S. Army Corps of Engineers Ohio River Division Laboratories performed tests on doweled, longitudinal construction joints at eight different Air Force Bases and at the Sharonville test facility (78, 79, 80). Results from the tests showed that the average load transfer for doweled longitudinal construction joints subjected to static loading was twenty-eight percent. Load transfer characteristics of these points were not affected by the side on which the joint was loaded. Poor construction technique and inadequate inspection during the construction of doweled joints was a significant factor affecting the load transfer capability. The particular method of dowel installation i.e., drilling and grouting or placement in plastic concrete did not appear to have a

significant affect on the load transfer capability of the dowels (79, 80).

For all pavements evaluated, the concrete was placed in fixed forms. Some problems of edge slumping and improper alignment have been encountered when placing dowels in plastic concrete with slip form pavers. The most satisfactory, and most expensive, method of dowel installation to date is to drill into the hardened concrete and to grout the dowels in place with an epoxy grout (26).

IV-C. Butt Joints

Butt Joints, tied or untied are the most popular type with contractors and paving engineers. The simplicity of construction and absence of construction related problems makes this type joint among the cheapest to construct. The problem then is to evaluate the performance potential of this type of joint for airfield pavements.

Two factors affect the load transfer potential of butt joints: 1) The size of joint opening and 2) the degree of aggregate interlock. To take maximum advantage of the low cost of construction with this type of joint, thickened edge pavements, sleeper slabs and other load transfer aids are not considered here. Only slabs of uniform thickness with and without ties are considered. Uniform support by stabilized subgrade are considered however.

Because of the problems experienced with the performance of keyed longitudinal joints the Navy has recently adopted a butt joint design without ties for its concrete pavements (74). The concrete slabs are always supported on a high quality stabilized base to provide partial load transfer. To date, after up to 8 years of service with 24 different pavement sections they have experienced no distress with this type of longitudinal joint construction (74).

It must be noted, that the pavements placed are generally not designed for heavy aircraft loads as the heaviest pavement sections are 10 inch concrete slabs on 6 inches of high quality cement treated base. Despite this excellent record, it is impossible to determine from the data available how these joints will continue to perform with time and when subjected to heavy aircraft gear loads such as provided by the DC-10's, L10-11's and 747 aircraft. This is a joint configuration which bears further evaluation and analysis.

Tests at PCA laboratories have shown that the load transfer between slabs is greatly affected by aggregate interlock and joint spacing (69). Beams with roughened faces to produce friction to simulate aggregate interlock were tested at various joint openings both with and without ties. For the tests where ties were used, deformed bars were installed at both mid-depth and at the one-third points of the test specimens top and bottom both singly and in pairs.

Results of static load tests on the slab specimens with roughened faces and six inches thick, tied with smooth No. 2 bars top and bottom were as effective in transferring loads across the joint as any system tested (keyed joints and doweled slabs). Under repetitive loading with slabs on a low foundation modulus, the tied slabs with aggregate interlock provided by the No. 2 bars was rated consistently more effective than untied keyed systems or doweled joints.

Further tests indicated the performance of the tied systems is closely related to the amount of steel used. Slabs with aggregate interlock but without the tie steel to prevent joint opening and smooth faced joints without aggregate interlock, even when tied, performed poorly. The slabs with aggregate interlock but untied and smooth joints with ties performed equally well when both were tested on a cement treated subbase (69). Care must be taken with this concept, however, as if the loads are very heavy are likely to quickly destroy the load transfer capability of the relatively thin cement treated subbase sections under these slabs.

Repetitive plate load and field studies by PCA (69) on the effectiveness of aggregate interlock as a load transfer mechanism illustrates the importance of joint openings on this factor.

Concrete slabs tested in this study were 46 in. (1.2 m) wide and

18 ft. (5.5 m) long with a transverse joint at the slab midpoint. The maximum aggregate size was 1 1/2 in. (38 mm). The joints were plane-of-weakness type formed by an insert 1 in. (25.4 mm) high in the bottom and a 1 inch (25.4 mm) deep groove in the top forming a roughened interface depth of 5 in. (127 mm) for 7 in. (178 mm) thick slabs and 7 in. (178 mm) for 9 in. (229 mm) thick slabs (100).

Joints were tested under repetitive loadings at openings of 0.035 in. (0.89 mm) to 0.085 in. (2.16 mm). It was determined that joint effectiveness decreased as the joint opening increased (See Figures 4 and 5). With an initial opening of 0.065 in. after one million, 9-kip (4082 kg) load applications reduced joint effectiveness to 19 percent of its original effectiveness. When the joint width was held constant, effectiveness of the joint decreased as the number of load applications increased. Joint effectiveness increased with increased foundation support and greater particle angularity (100).

Slabs with small joint openings did not lose effectiveness as rapidly as those with greater openings. This demonstrates the effectiveness of ties in providing good performance of joints.

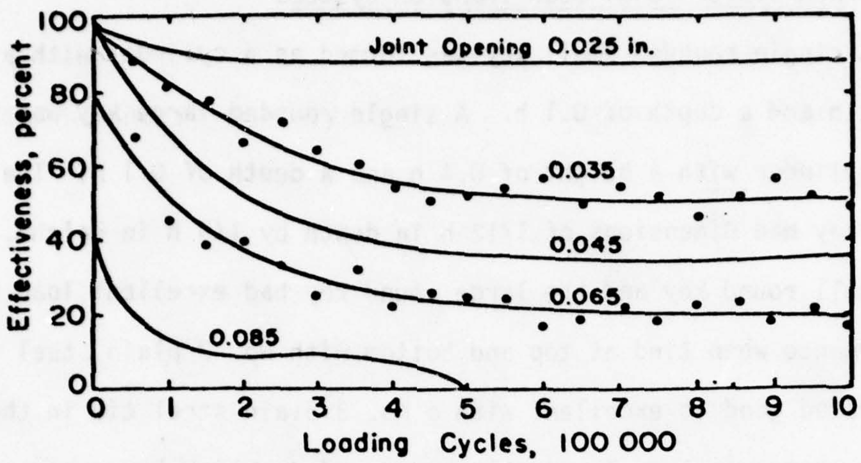


Figure 4. Influence of Joint Opening on Effectiveness, 9 in. Concrete Slab, 6 in. Gravel Subbase (Ref. 100)

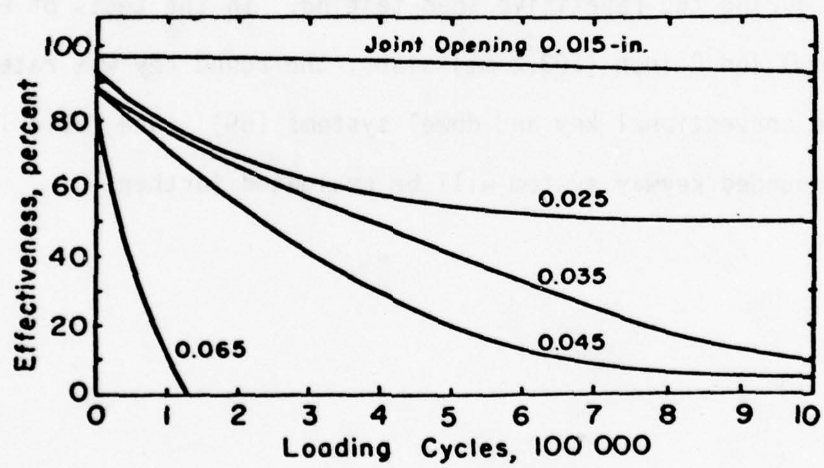


Figure 5. Influence of Joint Opening on Effectiveness, 7 in. Concrete Slab, 6 in. Gravel Subbase (Ref. 100)

IV-E. Non-conventional Load Transfer Systems

A single rounded small key was formed as a cylinder with a height of 0.2 h and a depth of 0.1 h. A single rounded large key was formed as a cylinder with a height of 0.4 h and a depth of 0.1 h. The double round key had dimensions of 1/12 h in depth by 1/6 h in height. Both the small round key and the large round key had excellent load transfer performance when tied at top and bottom with No. 2 plain steel ties. They were rated good to excellent with a No. 3 plain steel tie in the center during static tests. The double round and double-V keys were rated poor to inferior (69).

The round key tied with No. 2 steel ties gave excellent load transfer results during the repetitive load testing. In the tests of both 6 inch (152.4 mm) and 8 inch (203.2 mm) slabs, the round key was rated above both the conventional key and dowel systems (69) (See Table 1). The single rounded keyway system will be evaluated further.

V. Non-Conventional Load Transfer Systems

A major objective in the visits to the various airports was to discuss with the airport designers and maintenance personnel the problems encountered with the longitudinal construction joints and to get their suggestions for new or alternate load transfer concepts or systems. A number of alternate approaches was developed as a result of these discussions. Some of the more feasible of these suggestions are shown in Figures 6a-6c. These systems range from the simple butt joint with a stabilized subbase to transfer load to highly sophisticated mechanical transfer systems. Many of these suggestions are obviously impractical for general use but are included here because of their potential for limited use in areas where high performance may be more important than construction costs. Most of these suggestions or modifications were discussed with the airport engineers during the field visits with the results noted.

The efficiency of the alternate joint designs in terms of their load transfer capacity will be analyzed using a computer program developed for this purpose.[1] Based on this analysis, several of the most promising and potentially most cost effective will be selected for further evaluation. This evaluation will include detailed load transfer capability, cost and difficulties in construction and estimates of their potential. Field studies for validation of the most promising joints will be outlined in the final report.

Several non-conventional load transfer systems have been evaluated under laboratory test conditions. These will also be evaluated for their

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[1]Results from a preliminary evaluation of many of the alternate joint designs have been included as a separate report and were reviewed at the contract meeting on June 2, 1977.

potential as load transfer systems for use with slip form pavers.

The PCA tested the load transfer capability of different shaped keyways to determine which were capable of being formed with a slip form paver and which were also effective load transfer systems (69). Half rounded, double half rounded, and double "V" shaped were all evaluated as a part of this program.

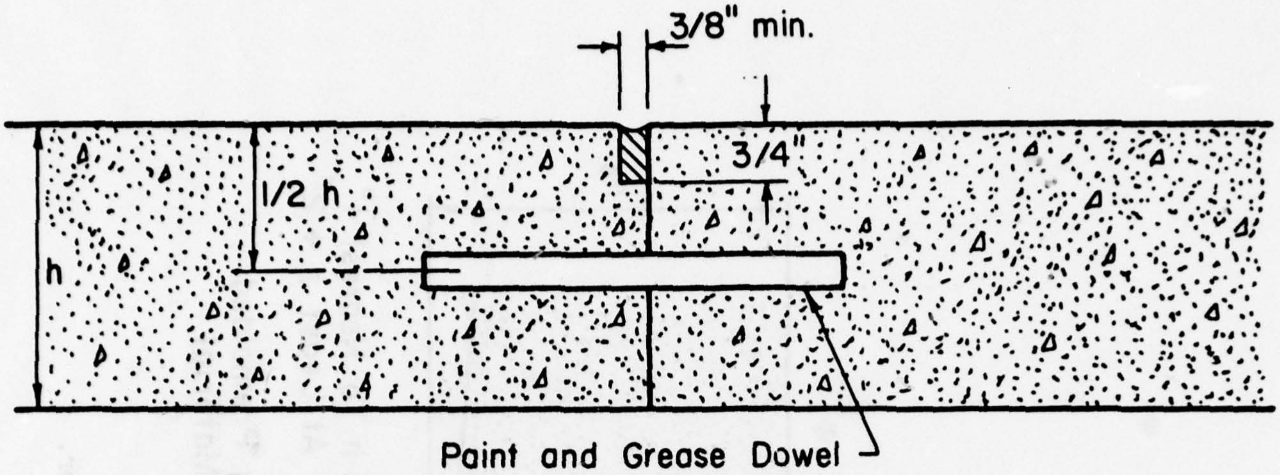


Figure 6a. Currently Used Longitudinal Joint - Doweled Load Transfer.

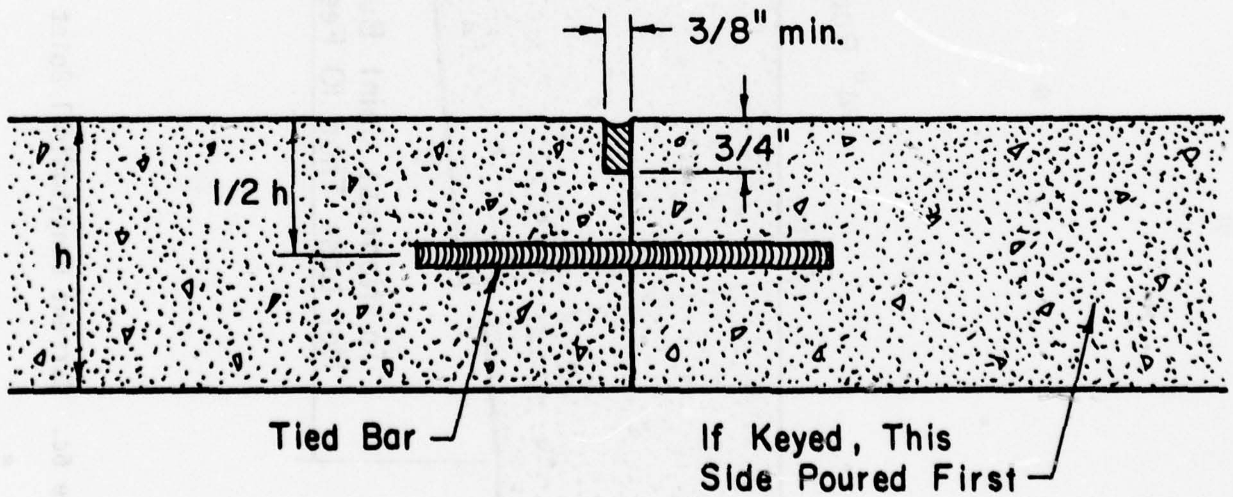


Figure 6b. Currently Used Longitudinal Joint - Tied Butt or Tied Keyed Joint.

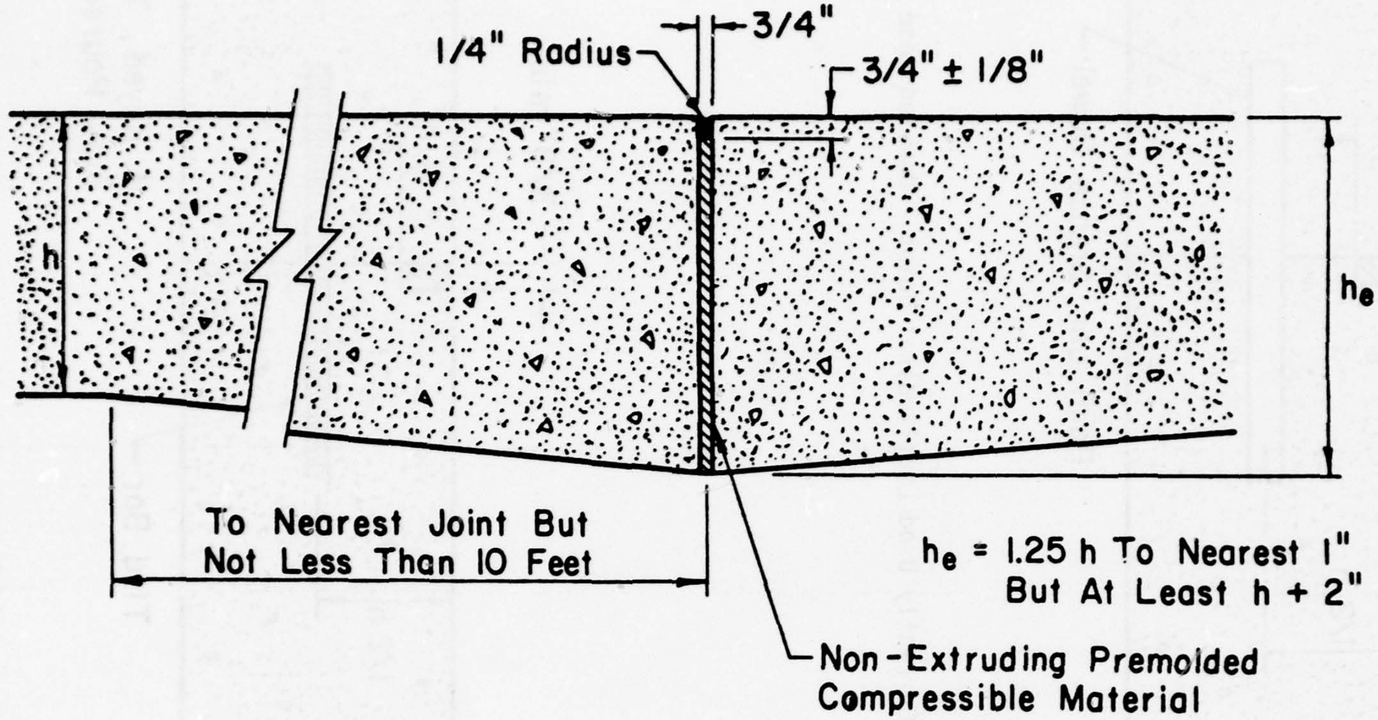


Figure 6c. Current Longitudinal Joint - Thickened Edge, No Load Transfer.

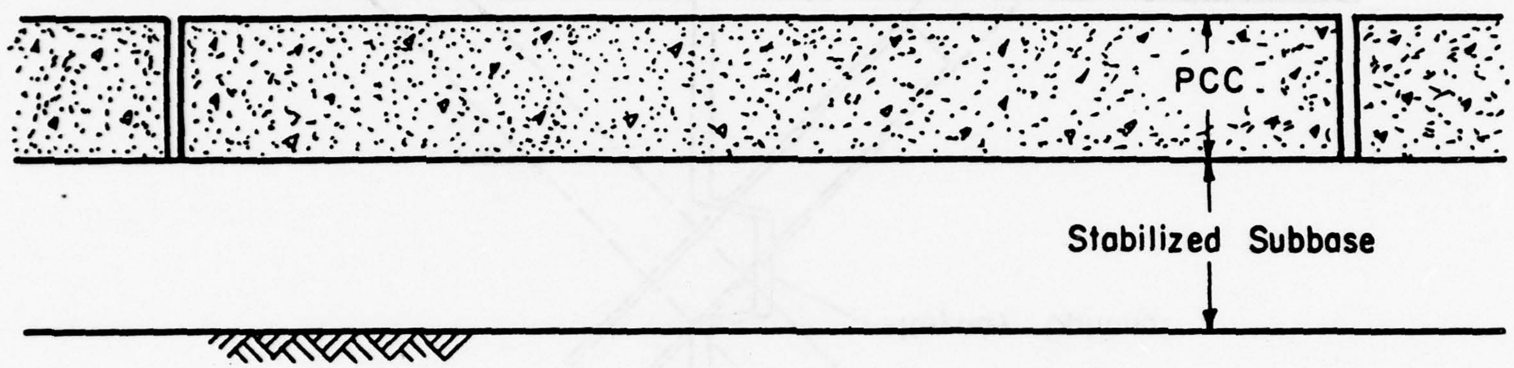


Figure 6d. Proposed Alternate Longitudinal Joint - Butt Joint No Load Transfer with Heavy Stabilized Subbase.

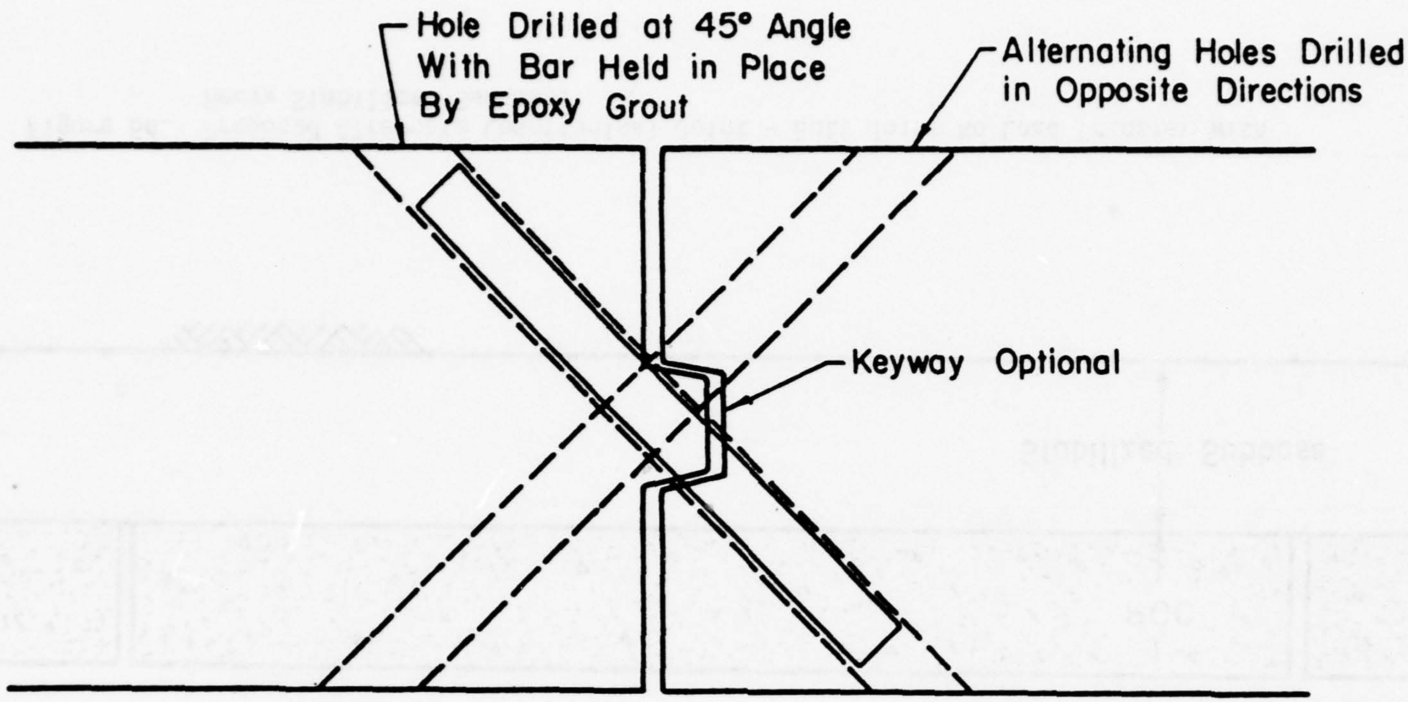


Figure 6e. Proposed Alternate Longitudinal Joint - Angled Dowel Load Transfer with Keyway Optional.

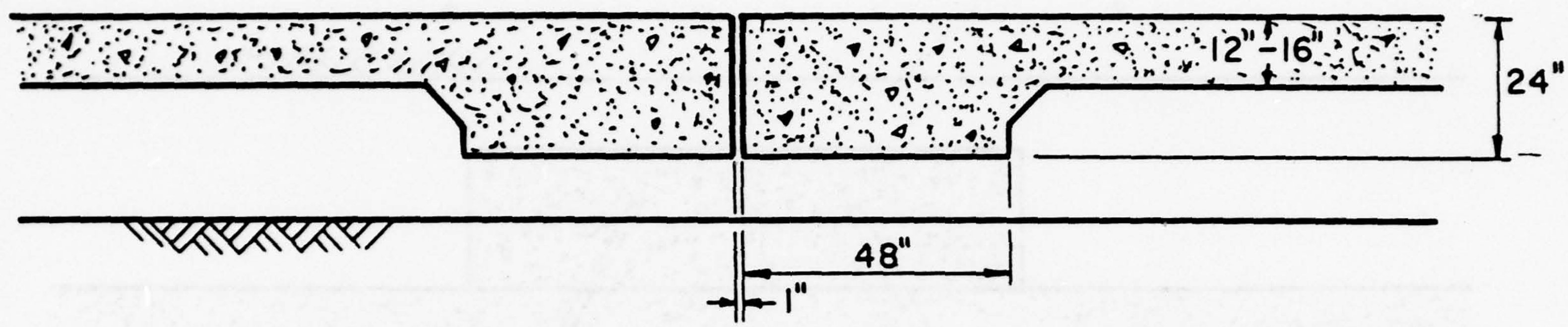


Figure 6f. Proposed Alternate Longitudinal Joint - Butt Joint, No Load Transfer with Thickened Edge.

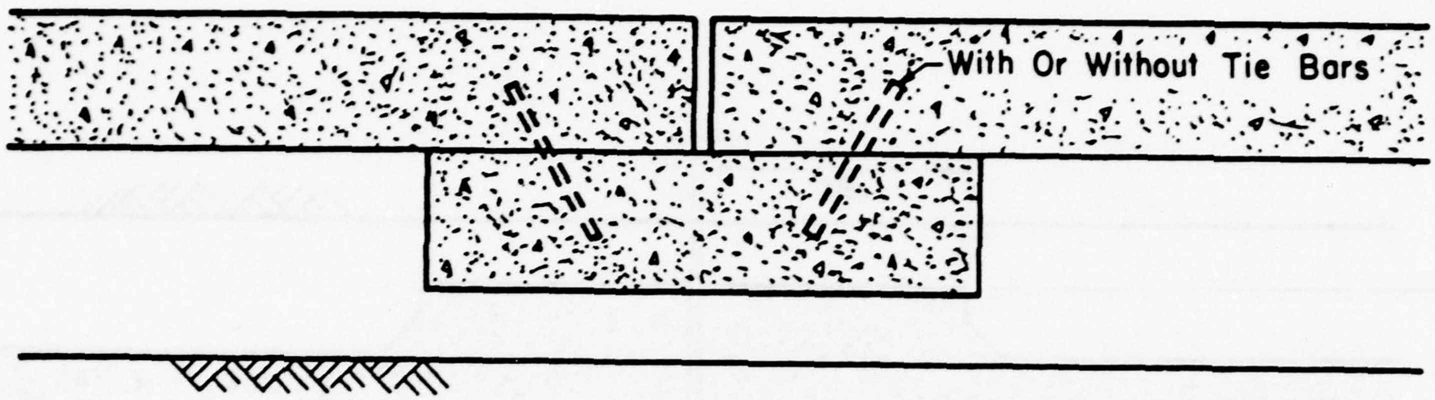


Figure 6g. Proposed Alternate Longitudinal Joint - Butt Joint Constructed on a Sleeper Slab for Load Transfer.

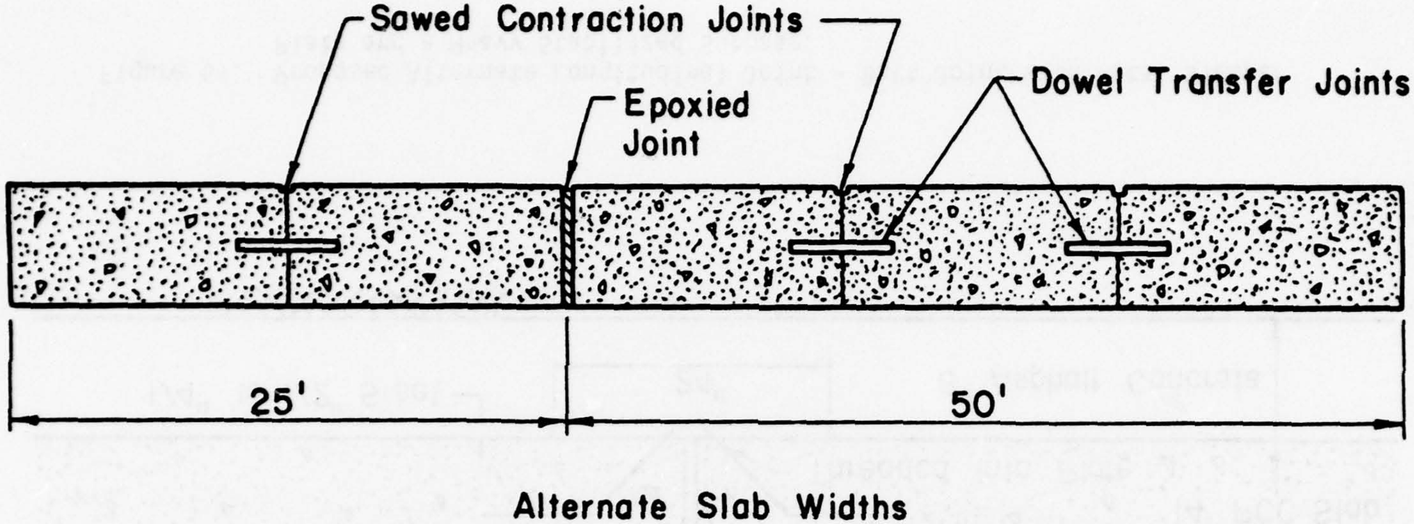


Figure 6h. Proposed Alternate Longitudinal Joint - Butt Joint Epoxied for Load Transfer with Doweled Hinge Joints at Mid-Points in the Slab.

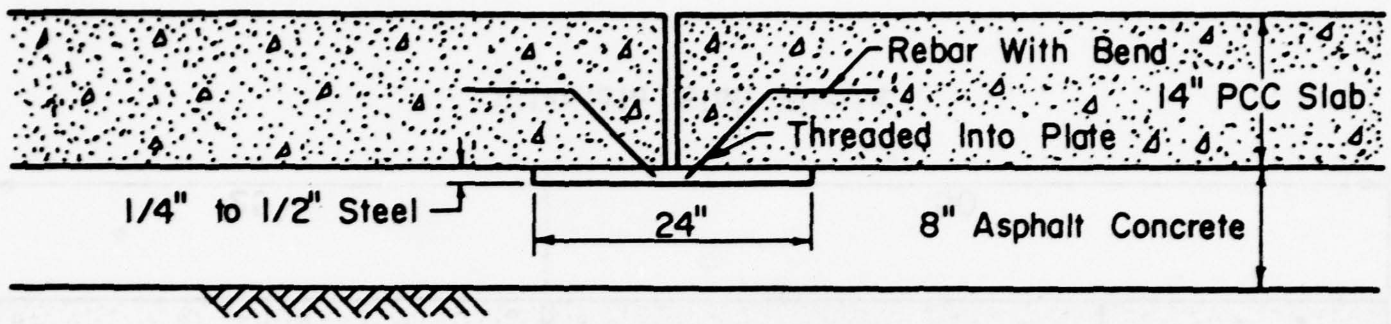


Figure 6i. Proposed Alternate Longitudinal Joint - Butt Joint with Metal Sleeper Plate and a Heavy Stabilized Subbase.

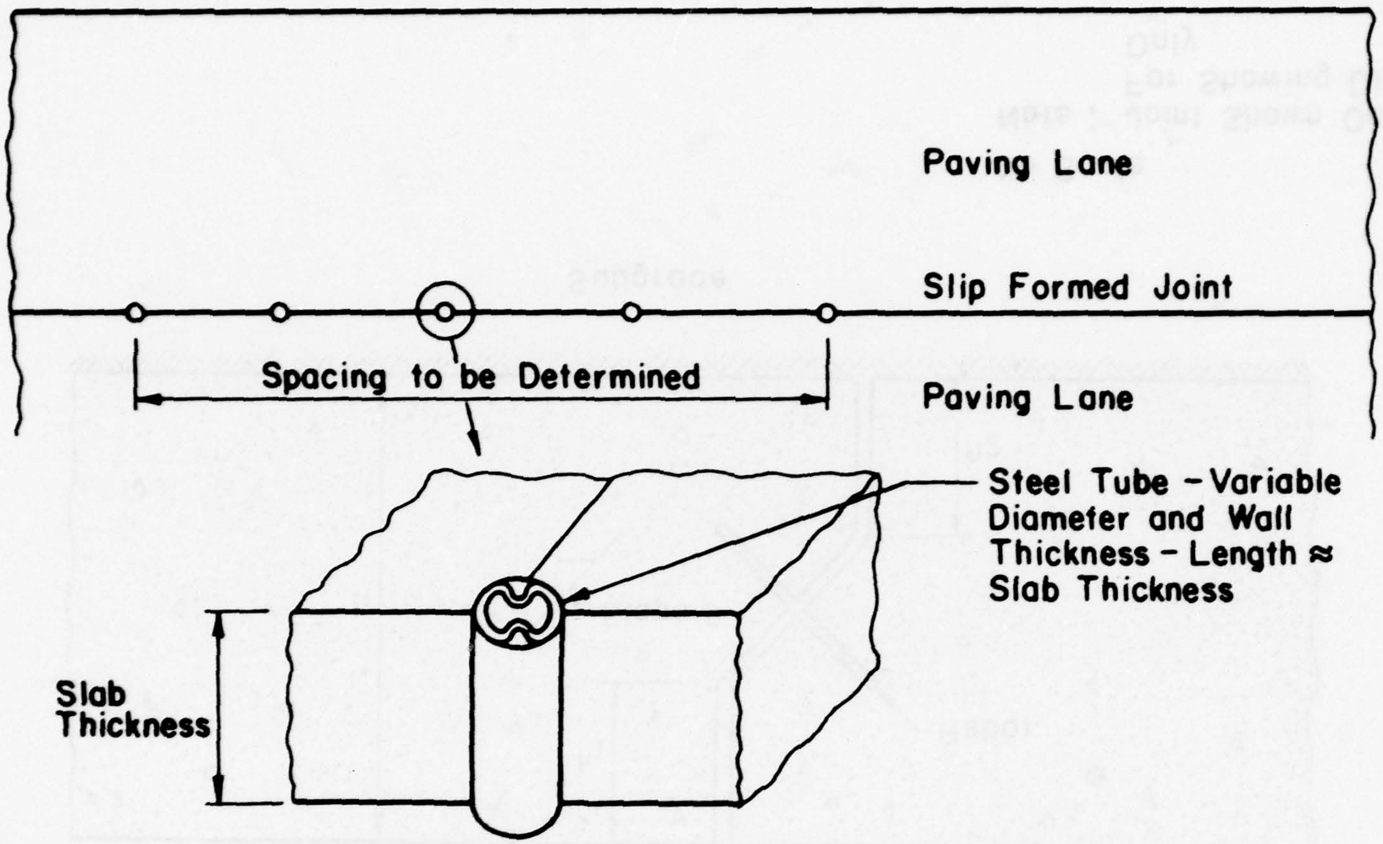
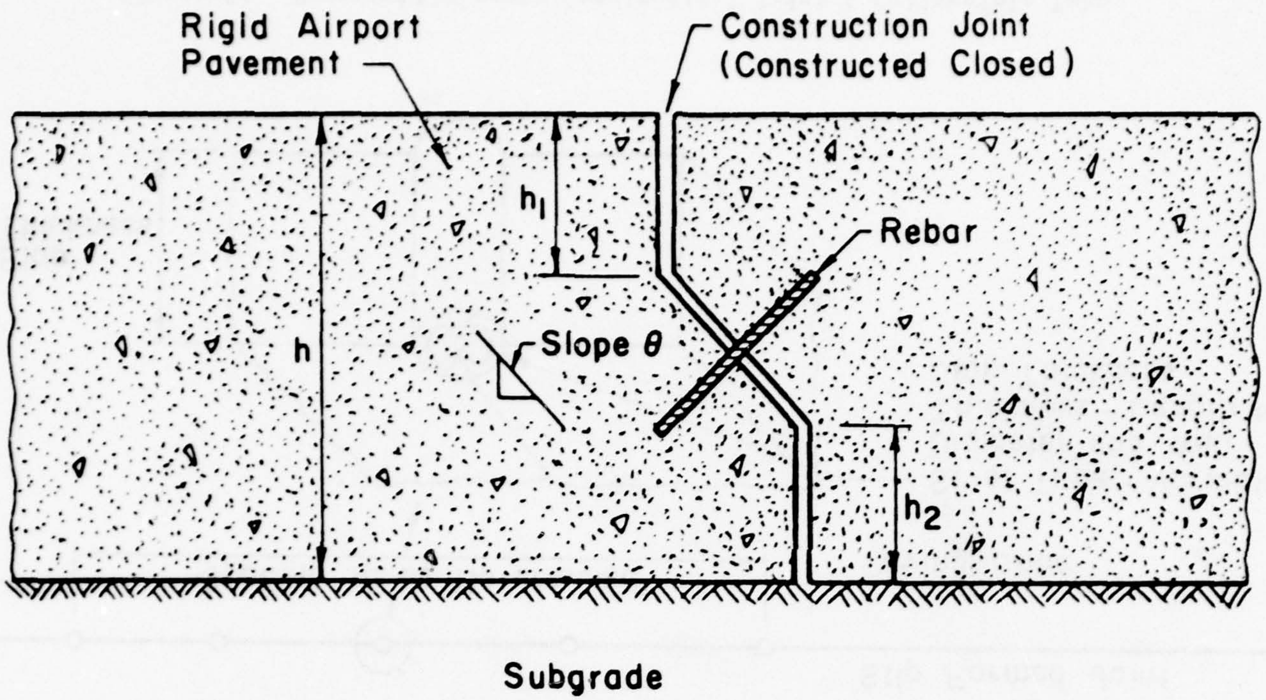


Figure 6j. Proposed Alternate Longitudinal Joint - Collapsible Tube Set Vertically in Joint (French Connection).



No Scale

Note : Joint Shown Open
For Showing Dimensions
Only

Figure 6k. Proposed Alternate Longitudinal Joint - Z-Joint Tied for Added Load Transfer.

VI. Non-Conventional Systems, Sealants, and Construction Techniques

This section contains reports of construction techniques and load transfer system approaches which might be useful in modifying present load transfer systems. In addition, non-conventional load transfer systems and joint sealants are covered.

VI-A. Precast Concrete Slab Pavements

To reduce construction time and to alleviate cold weather construction problems, it has been recommended that precast panels of PCC be used for construction of runways (15). In addition complicated load transfer systems could be formed at the precast site. Precast prestressed panels have been used in highway construction in South Dakota. The panels were interconnected with grout keys and tongue-and-fork connectors (49). Following emplacement, the panels were covered with a leveling layer of asphalt.

Mr. William Zuk has recommended that panels be cast with a tube implanted in one of the patterns shown in Figure 7. During emplacement, a cable would be threaded through the tubes to lock the slabs together for a load transfer system (50).

An experimental taxiway 1,150 ft (350 m) long and 75 ft (23 m) wide was constructed at the Melsbroek Airport, Brussels, Belgium, utilizing precast, prestressed slabs 39 ft (12 m) long, 4 ft, 1 in (1.25 m) wide and 3 in (100 mm) thick. The slabs were positioned with their length parallel to the center line of the taxiway, and the joints were caulked with mortar (See Figure 8). Transverse cables were threaded through ducts in the slabs and post-tensioned. Test loads of 50 tons (45.4 metric tons) revealed no weakness or discontinuities due to either the transverse or longitudinal joints (75).

The Michigan Department of State Highways and Transportation utilizes precast slabs to repair damaged highway sections in areas of high traffic density to shorten repair time. Dowels have been used in these for load transfer. Holes were drilled in the existing slabs and the dowels inserted. Following emplacement of the precast slab, the dowels were welded to metal plates cast in the precast slabs (52).

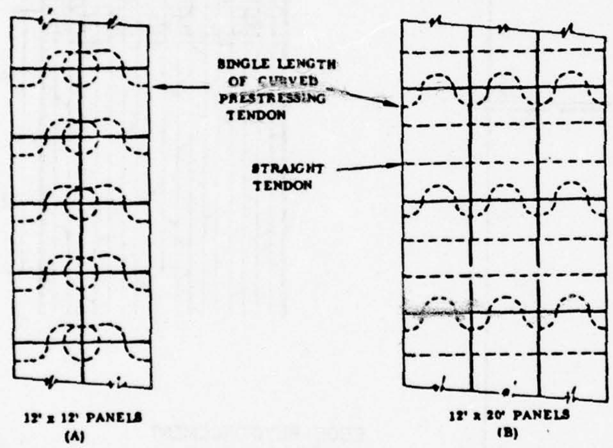


Figure 7. Plan View of Biaxial Compression Joining of Panels by Prestressing Tendons (Ref. 50).

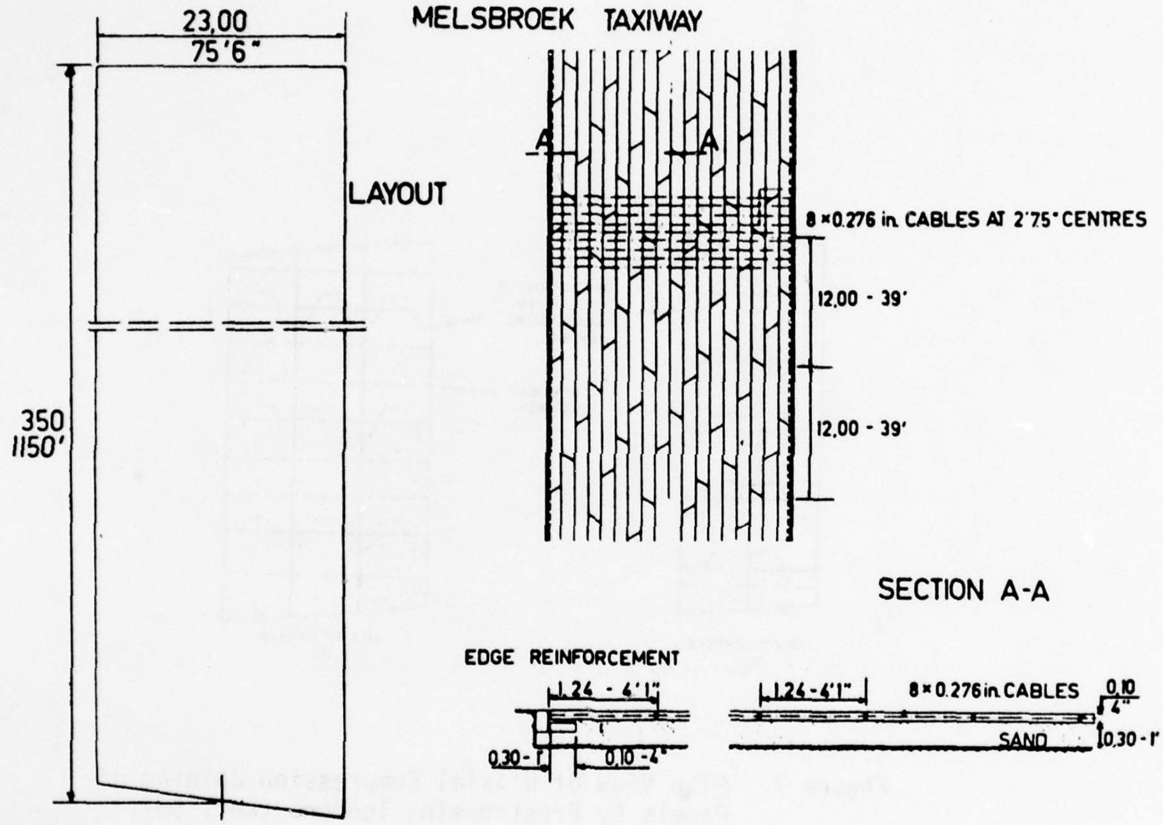


Figure 8. Layout for precast slab assembly - Melsbroek Airport (Ref. 75).

VI-B. Post-Tensioned Concrete Pavements

Prestressed concrete highway pavements have been constructed at Dulles International Airport and in Pennsylvania. The nominal 600 ft (183 m) length of slabs promises to eliminate many transverse joints and their associated problems. Since during construction both lanes were paved simultaneously by one slip-form paver, no longitudinal construction joint was made (53).

Two runways and one taxiway of the Maison-Blanche Airport in Algiers are constructed of prestressed concrete. The first runway and the taxiway were constructed in 1953-54. The second runway was completed in 1961. After more than seven years of high volume traffic, the pavements were considered in excellent condition with no indication of spalling or cracking.

The first runway is 8000 ft (2430 m) long, 197 ft (60 m) wide, and 7.2 in (18 cm) thick. The second runway is 7700 ft (2350 m) long, 148 ft (45 m) wide, and 6.4 in (16 cm) thick. The concrete slabs were prestressed in the transverse direction by post tensioning 12-wire tendons spaced at 4.36 ft (1.33 m) intervals. The pavements were prestressed longitudinally by post stressing with flat jacks and an elastic abutment. In the first pavement the jacking joints were constructed at 987 ft (300 m) intervals and at 645 ft (197 m) in the second. No information was provided on longitudinal construction joints (66).

The U. S. Army Corps of Engineers supervised the construction of a prestressed concrete pavement taxiway at Biggs Air Force Base, Texas, in 1959. Each of the three 500 ft (152.4 m) long by 75 ft (22.9 m) wide by 9 in (22.9 mm) thick sections was placed in three lanes 25 ft (7.6 m) wide. The longitudinal construction joints were roughened butt joints for

the full 9 in (22.9 mm) depth. The pavement was longitudinally and transversely post-tensioned, and the longitudinal joints were sealed with epoxy resin grout (81).

Static loading tests were performed on the taxiway using loaded B-52 aircraft in 1960. The tests indicated that the longitudinal construction joints had no appreciable effect on the strain measurements taken; however, they did influence the shape of the deflection curves from the tests. It was concluded that the prestressed runway could sustain in excess of 100,000 coverages of a B-52 aircraft at a gross weight of 525,000 lbs (238,136 kg) (82).

In 1966, Biggs Air Force Base was turned over to the U. S. Army, and a relatively small number of heavy aircraft have used it since. No specific traffic history for the prestressed runway was found; however, over 20,000 cycles of heavy aircraft have used the airfield since construction of the prestressed runway. Except for one patch, the prestressed PCC runway section was in excellent condition in January 1972 (83).

Two prestressed concrete test tracks (P1 and P2) were constructed by the U. S. Army Corps of Engineers in Sharonville, Ohio and tested to failure. In only one section of the 18 sections was the longitudinal construction joint the cause of failure. In this section only transverse reinforcing bars were used and the section had no transverse post-tensioning. The longitudinal construction joint did not fail in any section where transverse prestress had been applied. There was no indication that the type of longitudinal construction joint used had any appreciable effect on the performance of various test items where transverse prestress had been applied (91).

It was concluded that the presence of longitudinal construction

joints in the traffic area of prestressed pavements do not appear to have a significant effect on the performance of the pavement (91).

The following observation concerning longitudinal construction joints was made by Renz and Williams:

> "Where transverse prestress is used, the presence of a longitudinal construction joint does not affect the strength of the prestressed pavement materially. Special treatment of the joints does not appear to be necessary."[2]

It should be noted that the test tracks at Sharonville and the taxiway at Biggs Army Airfield were constructed using conventional forming equipment rather than slipform pavers.

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[2]Pg. 92, (76).

VI-C. Forming Holes in Fresh Concrete

Equipment has been developed and patented in Great Britain for forming holes in fresh concrete (51). This could allow emplacement of dowels for load transfer systems; however, it has only been tested in making vertical holes in horizontal concrete slabs. Since vibration is used to assist insertion of the hollow tube into the concrete, it is felt that this procedure would seriously alter the edge of fresh concrete emplaced by a slip form paver if a horizontal hole was formed on the vertical face of the concrete slab.

VI-D. Joint Sealants

The problem of adequate joint sealants was considered in a study recently completed at Georgia Institute of Technology (60). In addition, a study is currently being conducted in Pennsylvania (61) to evaluate the effectiveness of current sealant technology and materials. Although satisfactory performance of polyvinyl chloride elastomeric sealants and preformed noeprene seals for periods in excess of ten years has been reported (62), the best sealants available require a 3 to 4 year periodic maintenance program (60), and it cannot be assumed that sealants in longitudinal joints will last beyond a life span of 8 to 10 years. Further research is necessary in this area as recommended by Barksdale (60).

VI-E. Epoxy Strengthened Joints

Epoxy-systems have been used to patch, fill, and repair cracks in bridges, dams, runways, and roof-decks (58). Such a system might also be used to strengthen load transfer systems by bonding the slabs together across the longitudinal construction joint.

Most authors agree that epoxy-resin systems can be used to bond hardened concrete to either hardened or fresh concrete (58, 57, 59, 63, 64). The effective strength of the adhesive is reported to be the actual strength of the concrete since the strength of the adhesive and its bond is substantially higher than the cohesive strength of the concrete mass (63).

Cracks in the Walnut Lane Memorial Bridge in Philadelphia were repaired with epoxy resins. The epoxy resin was forced into the cracks under pressure in 1969. Tests using double the design live load on the bridge following the repair and follow-up inspections in 1970 indicated that the bridge and repairs were performing satisfactorily (56).

Epoxy-resins have been used to bond precast reinforced concrete panels to increase the load carrying capacities of existing reinforced concrete beams (57). The system provided a bond such that the entire built-up section acted as a monolithic structure. The author recommended that recognized manufacturers be contacted for information on bonding strengths (57).

It has been reported that inadequate surface preparation can substantially decrease the strength of the bond. Johnson (64) recommends that concrete surfaces be sand blasted and cleaned with high pressure air prior to applying the epoxy-resin to achieve an adequate bond. As

a result, the effectiveness of an epoxy resin injected into an existing construction joint as a load transfer joint is questionable. In addition, epoxy bonding would inhibit necessary hinge action.

F. Materials

One method of improving load transfer systems would be to improve the materials used in current systems. If the paving material in and around the keyway system was strengthened, the keyway system could possibly function without shear failure. One of the most promising materials which could be used in this manner is polymer modified concrete.

Polymer materials have been added to Portland Cement Concrete (PCC) in an attempt to overcome the shortcomings of PCC and to increase its versatility. Steinberg, Kukacka, Romano, and Dikeou (18, 37, 42, 43) have pioneered the study of polymers in concrete at Brookhaven National Laboratories (BNL). Their test results indicate dramatic changes in the material properties of PCC through use of polymers.

The different uses of polymers and methods of polymerization of PCC studied at BNL and elsewhere are listed below:

1. Polymer Impregnated Concrete (Full Depth) (1, 8, 27, 37, 39, 41)
2. Polymer Cement Concrete (1, 18, 27, 37, 41)
3. Polymer-Portland Cement Concrete (18, 27, 37)
4. Surface Coating Polymer Impregnated Concrete (Partial Depth) (10, 41)
5. Epoxy/Latex Modified Concrete (1, 17, 27)

The materials studied in modification of concrete are listed below:

1. Methyl Methacrylate (MMA) (10, 22, 41)
2. Isodecyl Methacrylate (10)
3. Isobutyl Methacrylate (10)
4. Styrene (22, 41)
5. Vinyl Acetate (22, 39)
6. Chloroprene Latex (1)

7. Styrene Butadione Rubber Latex (1)

8. Polyacryl Ethyl Ester Latex (1)

The first five of the above are classified as monomer systems for polymerization while the last three are used in epoxy/latex modification of concrete.

VII-A. A Polymer Impregnated Concrete

One of the principal means of creating polymer impregnated concrete (PIC) is full depth impregnation of hardened PCC with a monomer, and allowing it to subsequently polimerize. PIC is similar in appearance to ordinary PCC. The polymer material distributed through the PCC. In voids, pores and micro fractures forms a polymer matrix encasing the PCC (41). The new PIC has impressive properties compared to ordinary PCC (37).

1. Compressive strength increases from about 5000 psi (34.5 mn/m^2) to about 20,000 psi (137.9 mn/m^2), depending upon the percent of polymer.
2. Tensile strength also increases; as it remains at about 9% to 15% of compressive strength.
3. Capacity to endure freeze-thaw cycles improves immensely. While PCC specimens had failures (failure defined as 25% weight loss) during less than 700 freeze-thaw cycles, a similar PIC exhibited no appreciable decay after 3600 cycles. The reduction of water absorption capacity of PIC is credited for this property.
4. PIC exhibits zero creep.
5. PIC displays a doubled or tripled modulus of elasticity compared to the modulus of the original PCC. It exhibits linearity for up to 75% of failure strain.
6. Shear strength of PIC was not directly investigated.

Although the material properties of PIC are impressive, production of PIC in the field presents substantial problems. Probably the first problem encountered is a means of impregnation of the PCC with the monomer. Polymer impregnation occurs through the capillary pressure of

monomer and externally applied pressure. Capillary pressure alone produces sufficient impregnation to a useful depth; however, application of external pressure expedites the process. When no external pressure is applied, i.e., simple soaking, experiments by Sopler, et al., (35) disclosed that at times of 10, 100, and 1000 minutes, MMA reached a depth of impregnation of 0.315 in (0.8 cm), 0.75 in (1.9 cm) and 1.77 (4.5 cm) respectively. Impregnation will continue after 1000 minutes, but at a very slow rate. By applying 100 PSI (0.61 mn/m^2 G) pressure, penetration achieved in less than 10 minutes was equivalent to that achieved in 1000 minutes with no pressure (37).

The time required for satisfactory impregnation is affected by the routes the monomers must follow. Since the monomer penetrates the PCC through a series of micro-pores and micro-cracks, smaller pores and cracks require additional impregnation time. The size and continuity of pores is dependent on the air content. The length of time required to fill the pores and completely impregnate the concrete increases if the pores are pear shaped with a large enclosed volumes and a small connecting openings.

To attain maximum monomer loading, the mixture of gasses and water normally filling the pores of PCC must be displaced. Experiments demonstrate that evacuated specimens achieved a monomer loading of approximately 7 1/2% by weight while those with no means of evacuation achieved only 5 1/2% to 6% monomer loading by weight (18). For maximum loading, the gasses and water must either be removed prior to impregnation or escape routes must be provided during impregnation.

The absolute volume of pore spaces also affects the degree of monomer loading. Polymer loadings reach a higher percentage in PCC with a higher

percentage of entrained air since addition of air entrainment admixtures increases the amount of pore space. For example, entrained air percentages of 2%, 6% and 10% achieved a corresponding monomer loading of 5.9%, 7.2% and 9.5% by weight. The compressive strengths of the above products do not appreciably vary from 24,200 psi (166.9 mn/m^2) (18). One interpretation of this phenomenon suggests that higher polymer content compensates for PCC strength reduced by increased addition of entrained air. This is substantiated by the fact that low strength PCC (2000 psi) (13.8 mn/m^2) with high polymer loading (7.2%) exhibits a compressive strength of approximately 25,000 psi (172.4 mn/m^2) which is virtually identical to that of high strength PCC (10,000 psi) (68.9 mn/m^2) with low polymer loading (6.1%) (18).

To achieve maximum monomer loading, Kukacka and Romano (18) recommend the following processing techniques to overcome the aforementioned problem areas:

1. Oven-dry the PCC to constant weight at 302°F (150°C). Dryness should be achieved within approximately 24 hours.
2. Introduce monomer under vacuum at approximately 30 in (760 mm) Hg for 30 minutes subsequently pressurized to 10 PSIG (68,950 N/m^2 G) and maintain for 60 min.
3. Remove excess monomer.
4. Place specimen in hot water bath for polymerization.
5. Remove and clean specimen.

This technique presents obvious difficulties for use with large sections of concrete as well as for field application. Work at BNL continues research to overcome these difficulties.

Toxicity of monomer systems is a safety hazard which should be considered. Little information is available on specific monomer systems. Once

polymerized, all systems studied exhibited little if any toxicity and demonstrated long term chemical stability. If, however, polymers are subjected to high temperatures, there may be a chemical breakdown possibly releasing toxic vapors. Further studies of monomer toxicity are required (11). Until that has been accomplished, monomers should be handled with care.

Of the monomer systems studied, MMA, consistently out-performed the others in ease of application and versatility (10). As a result, most research projects utilized MMA impregnation.

VII-B. Polymer Impregnation Techniques, Field Methods

An entire bridge deck was impregnated with monomer on site in Denver, Colorado, during October, 1974, to evaluate field techniques for constructing Polymer Impregnated Concrete (PIC). Prior to impregnation, the PCC bridge deck (28 ft by 61 ft)(8.5 m by 18.6 m) was dryed under a portable enclosure by blowing hot air over the surface. Air temperature under the enclosure was maintained at 250°F (120°C) for about 3 days, until the concrete was completely dried to a depth of 3 to 4 in. (8 to 9 cm). The enclosure was removed, and the concrete was allowed to cool overnight (47).

The following morning, sand was spread over the bridge deck to a depth of about 1/4 in (6.35 mm) and thoroughly saturated with the monomer that was allowed to penetrate the concrete overnight. A polyethylene sheet was placed over the monomer-saturated sand to prevent evaporation of the monomer and a supply of monomer was added to sand reservoir to maintain a sufficient quantity for penetration. The drying enclosure was then placed over the monomer-saturated sand and polyethylene evaporation barrier, and hot air was blown into the enclosure at a temperature of about 160°F (70°C) to initiate polymerization. Cores taken from the bridge deck indicated a little more than 1 in (2.5 cm) polymer penetration (47).

Mehta et al. conducted laboratory experiments on impregnating PCC with methy methacylate (MMA) and other materials using patterned, flexible pressure mats. The liquid monomer was trapped in the cells of the mat and forced to impregnate the PCC by mechanical pressure.

Prior to impregnation, the concrete slabs were dried in an oven. A hydraulic tester was used to apply pressure to a steel loading plate slightly smaller than the mat. The plate applied pressure to the neoprene

rubber mat with diamond shaped pattern. A steel frame sealed and clamped to the slab was used as a diked reservoir. During each load cycle, the load was increased from 0 to 30 kips (133 kN) over a 15.5 x 15.5 in.(39.4 cm x 39.4 cm) area [or from 9 to 125 psi (0 to 0.86 MPa)] within 22 seconds and then released within 8 seconds. A 30 second break period before re-loading was provided to simulate passage of a roller over a particular cross section of a mat. After impregnation the surface of the slabs were coated with several pounds of cement and soaked with monomer. The slabs were heated to a surface temperature of 220°F (104°C). The polymer impregnation depth varied from 6 in (15 cm) in the center of the slab to 1 in (2.5 cm) outside the pressure mat area.

The researchers believe that the pressure mat technique would be more efficient in the field than in the lab. Three field techniques were recommended for study.

1. Place molded mat over monomer on a diked pavement section and apply pressure with a slow moving roller.
2. Cover the roller drums with a diamond patterned neoprene mat and roll pavement section within a dike system filled with monomer.
3. Use the roller with coated drums and force monomer into the cells from within the roller.

H. C. Mehta has also proposed a mobile pressure impregnation device for field impregnation of PCC and subsequent polymerization. He was able to dry a portion of a PCC bridge deck in sub-freezing temperature to a depth of 4 in to 5 in (100 mm to 130 mm) in 9 hours using a fixed propane torch assembly to elevate the temperature of the PCC to 250°F (121°C). He reports that it would be practical to dry an entire bridge deck to a depth

of 5 in (130 mm) using this method. A prototype pressure impregnation device was bolted to the concrete and sealed with a gasket. The monomer was placed in the device which spread in on the PCC slab and maintained it under pressure. Penetrations at 5 in (130 mm) were achieved in 17 to 24 hours under 30 PSI (210 kN/m^2) and in 2 to 3.5 hours under 80 PSI (550 kN/m^2). The device was then removed; the PCC was covered with monomer saturated sand and enclosed in a polyethylene film. Live steam was then played on it for 5 hours to polymerize the monomer. The PCC was penetrated and the monomer polymerized to a depth of 5 in. A conceptual model for field application is shown in Figure 9 (71).

A gas-fired infrared heater has been used to simulate point drying on a bridge deck. A metal impregnation box was then placed over the dried area and sealed to the concrete with a silastic rubber compound. The monomer system was placed in the box, and the box was covered with a polyethylene sheet. The system was allowed to stand for four days. At the end of this soak period, the excess monomer was removed, and 7 in (177.8 mm) of water was placed in the box. The temperature of the water was maintained at 185^{o}F (85^{o}C). Impregnation and polymerization to a depth of 4 in (101.6 mm) was achieved (72).

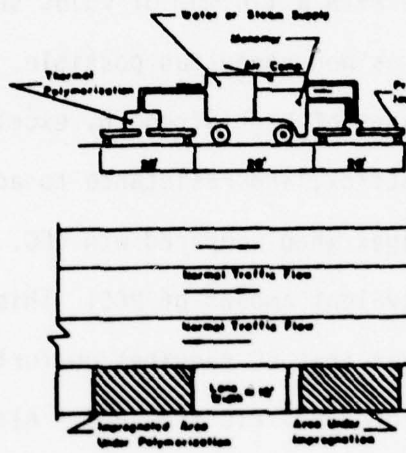

Figure 9. Conceptual Scale-Up of Pressure Impregnation Technique Used Herein (Ref. 71).

VII-C. Polymer Concrete

To overcome impregnation problems, polymers have been used as the cementing agent. This results in a different product known as polymer concrete (PC).

To form PC, aggregate is mixed with a monomer which polymerizes after the mixture is placed in forms and compacted. Compressive strength nearly equal to PIC have been achieved (41). Phenol-formaldehyde, furan as well as epoxy and polyester resins have been used (27). To prevent unnecessary use of expensive monomers, an aggregate selection which produced a dense mixture with a minimum of voids should be selected and the aggregate should be as non-porous as possible.

PC provides advantages of high strength, excellent durability, resistance to chemical attack, and resistance to acids; however, PC presents several disadvantages when compared wth PCC. PC may cost 5 to 30 times as much as an equivalent amount of PCC. This is compensated for to some extent by the fact that PC requires no further processing costs as would be required to produce PIC from PCC. Also, the strength of PC is considerably increased over that of PCC providing savings due to decreases in required thickness. When polymers are used as the only cementing agent, PC exhibits considerable creep due to its visco-elastic properties. It also exhibits a large degree of flexibility which may limit its usefulness for some applications.

VII-D. Polymer (Portland) Cement Concrete

As a compromise between the impregnation problems of PIC and the cost/properties problems of PC, the PCC and polymers were combined during mixing. The conclusion of Steinberg, who with others experimented with the process at BNL, are as follows:

> "The results obtained are either disappointing or relatively modest improvements of strength and durability. In many cases materials poorer than concrete are obtained. Under the best conditions compressive strength improvement over conventional concrete of ~50% are obtained with relatively high polymer concentrations of 30%. [For comparison, maximum polymer loadings of PIC observed are on the order of 10% and the average weight in PC is approximately 20% depending on exact design (24).] Polyester-styrene, epoxy styrene, furans and vinylidene chloride have been used with PCC with limited success. This is explained by the fact that organic materials are incompatible with aqueous systems and in many cases, interfere with alkaline cement hydration process (37)."

The concept of adding some type of admixture to PCC during mixing requires further study.

VII-E. Latex/Epoxy Modified Concrete

Polymer latexes and epoxies have also been added to PCC during the mixing process and subsequently polymerized. Of the materials investigated, polyacryl ethyl ester latex (PAE), styrene butadiene rubber latex (SBR) and chloroprene latex, only PAE exhibited useful properties as an additive. Material property tests demonstrated that at maximum compressive strength, the strain which occurred was substantially greater than for untreated concrete. When PCC was modified with PAE loadings of 10%, 20% and 30% by weight the strain at maximum compressive strength was 140%, 170% and 320% respectively of the strain for non-treated PCC. The increase in ductility was achieved at a decrease in ultimate compressive strength. For loadings of 15%, 25% and 30% the compressive strength was 81%, 72% and 49% respectively of that of non-treated PCC. The researcher suggested the possibility of producing a ductile modified concrete which possessed ample energy absorption under shock load (1).

VII-F. Fiber Reinforced Concrete (FRC)

Significantly improved material properties have been achieved by adding a small amount of relatively thin steel fibers to PCC. The reinforcing action of the fibers minimizes the effect of micro fractures and greatly reduces spalling of the concrete. Problems have been encountered in use of FRC due to "clumping" of the fibers during mixing as the fibers tend to "ball" together in intertwined clumps with a resultant uneven fiber distribution throughout the mix.

A construction project at Ft. Hood, Texas, overcame the clumping problem using the following techniques. A fiber dimension of 1/2 in (1.27 cm) long by 10 mills (254 microns) in diameter were used at 1.5% by volume of PCC to overlay 27,000 sq yd (22,575 m^2) by 4 in (10.2 cm) thick. A 8.5 bag mix with maximum 3/8 in (0.95 cm) aggregate PCC was used. The fibers were fed into the mix at the same time as the aggregate. The use of the short fibers and a high rate of mix speed tended to inhibit clump formation. A high slump, minimum of 3 1/2 in (11.43 cm) was used to facilitate lubrication.

The high slump may make the use of FRC mixed with these techniques unsuitable for use in slip-form paving. The project at Ft. Hood used conventional fixed forming.

VII-G. Vacuum Concrete

Vacuum treating concrete has been suggested as a technique for increasing the strength of concrete along longitudinal joints. No publication directly assessing this possibility was located.

The principal advantage of vacuum treated concrete is achieved by removing water from formed concrete. Engineers have applied this technique on floor slabs in Scandanavian countries (44); however, specific construction techniques were not reported in English literature reviewed by this author.

A vacuum has been applied to wet concrete while it was in the mixing cycle to reduce the total water content. Tynes (45) performed tests on vacuum mixed concrete in which he measured air content, slump, compressive strength, modulus of elasticity and freeze-thaw durability. His work revealed that compressive strength did not increase significantly, modulus of elasticity increased only slightly, and the slump was not affected by vacuum mixing. The reduced air content resulted in a significant reduction in freeze-thaw durability.

A potential problem in the use of vacuum techniques with slip form pavers is that of seizure of the plastic concrete on the forms. If such seizure did occur it would likely cause tearing of the plastic concrete.

VIII. Reinforcement of Existing Longitudinal Joints

Three mechanical methods of reinforcing longitudinal joints with keyway load transfer systems have been tested at WES. These were horizontal dowels grouted into kerfs cut perpendicular to the joint, angled dowels placed and grouted in cored holes at alternate 45 degree angles through the joint, and grouting in underreamed voids in the slabs. In addition, epoxy resins have been used to bond hardened concrete in other curcumstances and might also be used to increase the effectiveness of existing load transfer systems. Any method selected will be time consuming and expensive. Of those tested at WES, R. W. Grau concluded the angled dowels are the most economical and practical method (13). The first three reported methods were used to reinforce a keyed longitudinal joint in an 11 in (279.4 mm) thick PCC pavement constructed over a low strength subgrade of heavy clay. The keyway used conformed to U. S. Army, Corps of Engineers specifications previously and shown in Figure 3. All three of the methods performed satisfactorily according to Grau (13).

VIII-A. Horizontal Dowels

Dowels 1 in (25.4 mm) in diameter and 12 in (304.8 mm) long were placed in kerfs cut perpendicular to the longitudinal joint to be reinforced. The kerfs were 1 1/2 in (38.1 mm) wide, 4 1/2 in (114.3 mm) to 5 5/8 in (142.9 mm) deep, 28 in (711.2 mm) long on the surface, and 12 to 13 in (304.8 to 330.2 mm) long at the bottom of the cut. The kerfs were cut 12 in (304.8 mm) center to center in the 11 in (279.4 mm) thick PCC pavement. The dowels were bonded to the bottom of the kerf with epoxy mortar. One end of the dowels was greased to prevent bonding. The remainder of the kerf was later filled with an epoxy mortar (13).

The results of the test indicated that the strengthened keyway would last as long as the pavement. The bars were intact and still performing at the end of the test, and no joint failure was reported (13).

VIII-B. Angled Dowels

A truck-mounted drill rig was used to core 1 1/2 in (38.1 mm) diameter holes at a 45 degree angle on 12 in (304.8 mm) centers. Alternate holes were sloped in opposite directions and were located so that they would pass through the center of the keyway. A 1 in (25.4 mm) diameter by 12 3/4 in (323.8 mm) long dowel was grouted into each hole with an epoxy grout. This load transfer system performed satisfactorily for the life of the 11 in (279.4 mm) thick PCC pavement. At the end of the test the bars were intact and still performing satisfactorily (13).

VIII-C. Sand-Cement Grout in Underreamed Voids

Eight inch (203.2 mm) diameter access holes were drilled through the center of the longitudinal joint on 3 ft (0.91 m) centers. A void approximately 42 in (1.07 m) in diameter by 24 in (0.61 m) deep was created by an underreaming device through the access holes. The voids and access holes were filled with sand-cement grout.

During the test water squirted out around the edges of two of the access holes indicating that bonding between the grout and slab was very poor. Spalling and flaking around the edges of the access holes occurred by the end of the test. The longitudinal keyed joint displayed faulting of 3/8 in (9.52 mm) to 1/2 in (12.7 mm) in several locations by the end of the test; however, it was felt that this type of strengthening would perform as long as the pavement remained in serviceable condition (13).

IX. Summary and Conclusions

Construction of keyed longitudinal joints with slip form pavers is a primary cause of paving problems on airport pavements. If keyed joints are constructed with slip form pavers, it is much better to form the female side of the joint first and to use metal insert to form the slot thereby reducing edge slump. A few instances were encountered in which the male side of the key was successfully formed without significant problems, but such conditions were rare. In such instances, the concrete mixing and paving operations were under control of the same organization, and a very high level of quality control employed in mixing and placing of the concrete. In general, few contractors can consistently form the male side of the keyway first and expect not to have considerable sloughing of the keyway. Installation of tie bars and/or dowels into the keyway tends to significantly magnify the sloughing problem.

All contractors and field engineers interviewed in this study indicated a strong preference not to construct longitudinal joints with keyways.

Even when constructed with an apparent minimum of problems, keyways tend to cause maintenance problems. When used in relatively thin pavements (less than 14 inches) and loaded with heavy aircraft, there is a tendency for the keyways to fail with the concomitant spall of large pieces of concrete. Also, since use of the keyways produce areas of high stress concentration, any deterioration of the concrete will be reflected in premature deterioration of the concrete near the keyway.

Keyways in thick slabs or thickened edge pavements (greater than 14 inches) appear to perform in a satisfactory manner if properly installed. This has led the project staff to conclude, somewhat facetiously

perhaps, that keyed joints perform well as long as they are not needed. When really needed as a load transfer device, however, the record of performance is not good. Thus, use of keyed joints should be eliminated if a satisfactory alternate load transfer system can be found.

Some alternate joint conditions and load transfer systems which have been used, or at least tried, include butt joints with and without thickened edges, doweled butt joints, and tied butt joints. Based on interviews with contractors and field engineers, it is believed that all of the above can be constructed with a minimum of construction problems. The problems are to evaluate the load transfer capacity, the potential performance and cost of construction of these systems. As a part of the recommended analysis, modifications to these systems can be developed which will improve the potential load transfer capacity and performance.

As was suggested by several engineers and contractors, the easiest and cheapest longitudinal joints to construct, especially with slip form pavers, are butt joints without ties, dowels or keyways. Drilled dowels installed in precast butt joints are generally considered to be easy to construct, but are also quite expensive. Since the original field survey for this study was made several instances of drilled dowel installation have been found which confirm the conclusions on ease of construction and cost. It is noted, however, that these doweled joints may have been overdesigned and that the cost may be significantly reduced by optimizing the design of the dowels in terms of performance as affected by dowel size and spacing.

Based on the findings from this survey it is concluded that more cost effective longitudinal joints can be found for use with slip formed concrete pavements if the keyway systems are replaced by other load transfer systems.

Some recommendations for design and validation of these systems are presented in the Section X of this report.

X. Recommendations

Based on the findings from this study the following recommendations are given for further study on the next phase of this study:

1. To quantify the relative cost effectiveness of current and proposed load transfer systems for longitudinal joints used in concrete pavements especially those constructed with slip form pavers.
2. To select 3 or 4 of the most promising, i.e., potentially the most cost effective, systems for further study and optimization.
3. Make detailed analyses of several of the most promising load transfer systems including the butt jointed system with and without dowels for potential performance and cost.
4. Develop design procedures for one or two of the most promising of the joint systems analyzed and establish validation procedures for these joints.

Items 1 through 3 and part of Item 4 will be accomplished as part of the next phase of this study. Field verification of joint performance is not a part of this study.

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Baton Rouge Airport

The visit to Baton Rouge Airport was undertaken to view some unusual construction details with regards to load transfer systems and some distress associated with installation of the load transfer systems. Since the pavement engineer for the airport (a consultant) chose not to meet with the FAA Representative Mr. Blair Harvey and me during our visit, not all details for the design and construction are available on these pavements. The maintenance foreman, Mr. Tom Gaudeau, for the pavements did accompany us on the pavement inspection and supplied most of the information reported.

These pavements are non-reinforced concrete with transverse joints at 15 foot spacing. Star lugs similar to those manufactured by Texas Foundaries and shown in Figures A-1 and A-2 were used as the load transfer devices in the transverse joints with a metal joint former placed in the plastic concrete in lieu of sawing the dummy joints. This was necessary because the hardness of the aggregates made sawing impractical. Longitudinal joints were slip formed without keyways but with dowels inserted into the plastic concrete. Dowel size was not known. A typical longitudinal joint with dowels is shown in Figure A-3.

No particular or significant problems were observed with the longitudinal joints used. Some corner distress was seen at the intersection of the transverse and the longitudinal joints as shown in Figures A-4 and A-5. This corner distress is believed to have been due to improper placement and improper timing in the placement of the metal joint former. Phone conversations with the consultant who designed and supervised the construction indicated that after the timing and correct procedures for installation had been worked out, no further distress of this type was observed. Figures A-6 and A-7 show some of the construction sequence in the installation of these joints.

In general, the construction at Baton Rouge indicated an excess of hand working. If the paving machines had been allowed to finish the concrete without benefit of the added hand working, a more uniform and smoother looking job would likely have resulted.

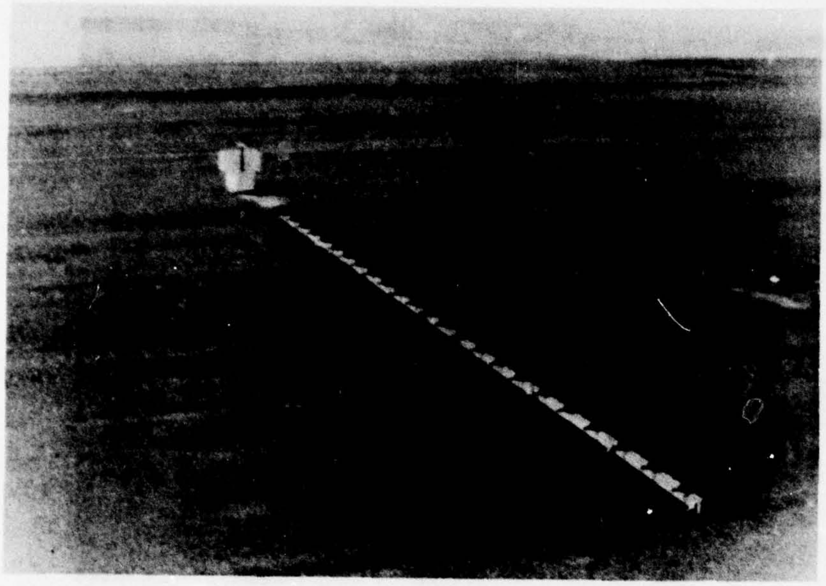

Figure A-1 - Starlug Load Transfer Device Used in the Baton Rouge Airport (B. Harvey)

Figure A-2 - Starlug Load Transfer Device Installed and Ready For Cover (B. Harvey)

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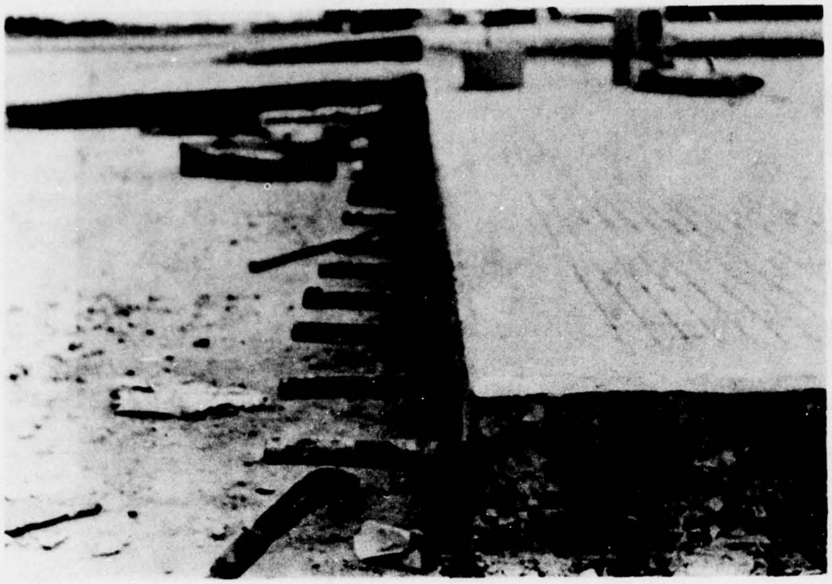


Figure A-3 - Longitudinal Joint With Dowels From Baton Rouge Airport (B. Harvey)

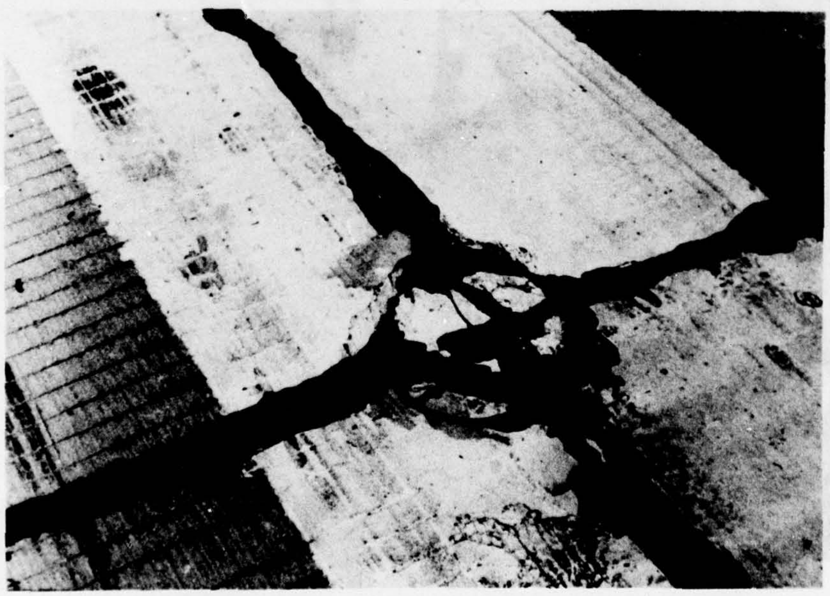


Figure A-4 - Corner Distress in Baton Rouge, Airport

Figure A-5 - Distress Along Transverse Joint in Baton Rouge Airport

Figure A-6 - Installation of Transverse Joint Former Baton Rouge Airport (B. Harvey)

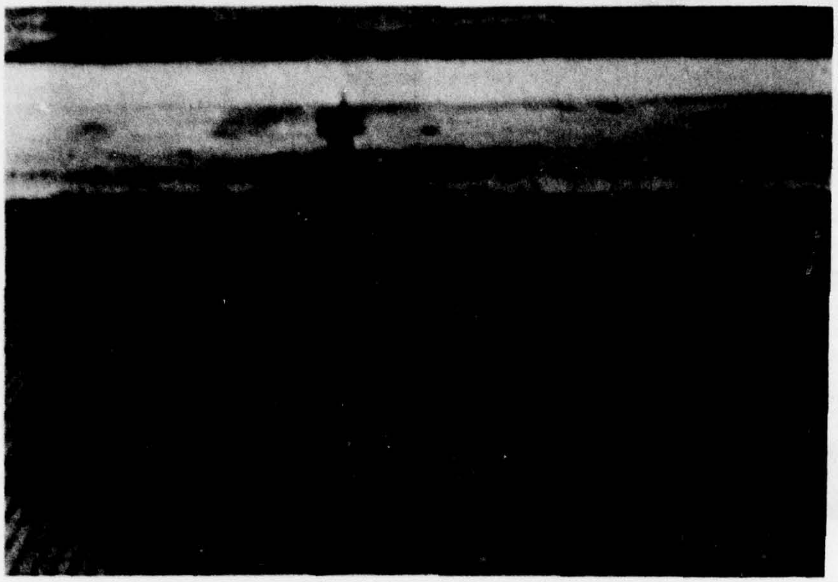


Figure A-7 - Completed Transverse Joint, Baton Rouge Airport (B. Harvey)

Midway Airport Chicago

The runways at Chicago Midway Airport were concrete pavements overlayed with concrete. Typical overlay thicknesses were 8 inches with keyed longitudinal joints.

Keyway failures in the longitudinal joints were noted shortly after completion of the overlay and progressively worsened. Figure A-8 shows a keyway failure at an early stage in its development. With time the failure will lengthen and eventually spall. Concrete spalls were observed along the keyed joints. Many of these spalls had been repaired at the time of this inspection.

According to the airport engineers, repairs on the failed keyways consists of first sawing off the keyway tongue with a diamond bladed saw. A second cut, as shown in Figure A-9 is then made to a depth of the upper keyway failure. The loose concrete is removed and the resulting trench filled with an asphalt concrete. Figures A-10 thorugh A-12 show typical repaired areas ranging from isolated failure patterns (Figure A-10) to continuous repairs for substantial lengths of joint (Figure A-11 and A-12). Figure A-11 shows a repaired joint with additional distress evident near the end of the repairs.

Most repairs have performed very effectively over a number of years with no loss of asphaltic concrete from the repairs, and no spalling of the AC or the adjacent concrete. These results suggest that the keyways were in fact not needed for the initial pavement design. Traffic on these pavements were primarily general aviation aircraft with some commercial (727 Max) aircraft. All distressed areas shown were along the centerline of the runway where traffic loads are expected to be fairly light.

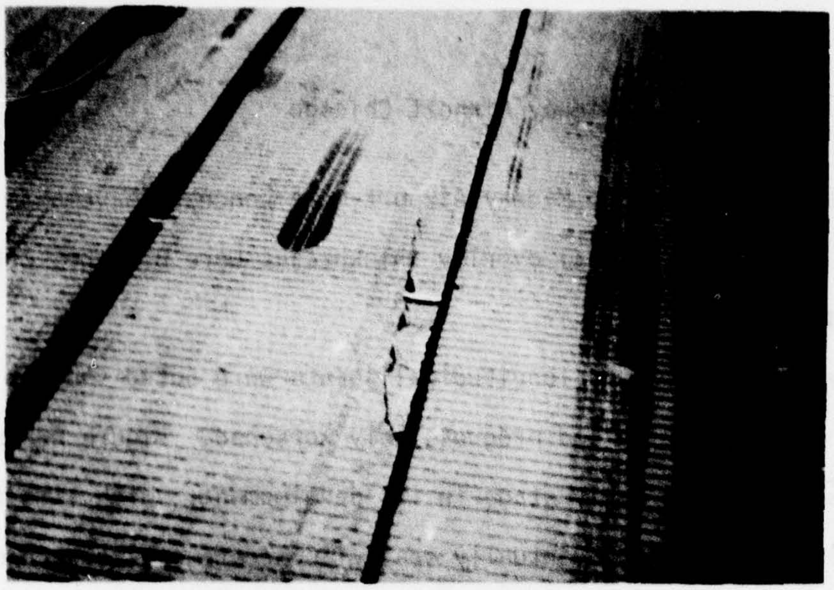


Figure A-8 - Initiation of Keyway Failure, Midway Airport

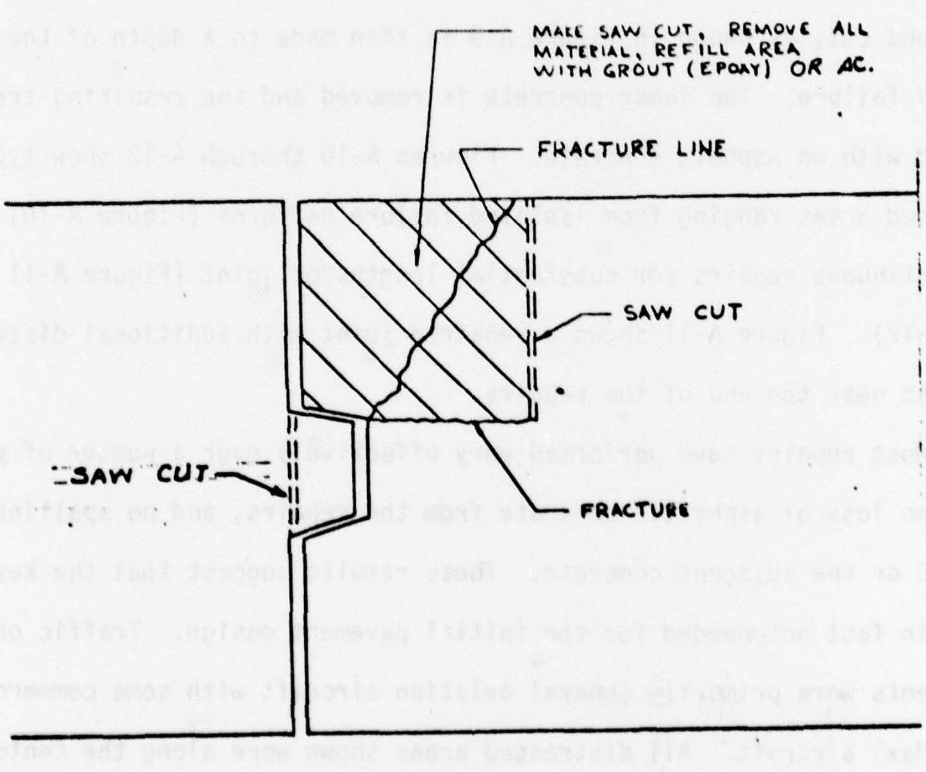


Figure A-9 - Sequence of Saw Cuts Used in the Repair of Distressed Longitudinal Joints at Midway

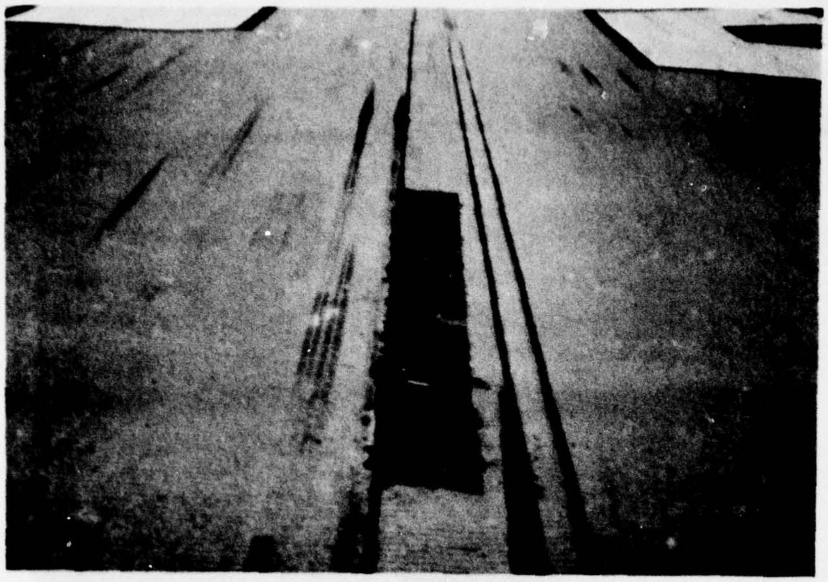


Figure A-10 - Isolated Patch of Distressed Longitudinal Joint Midway Airport

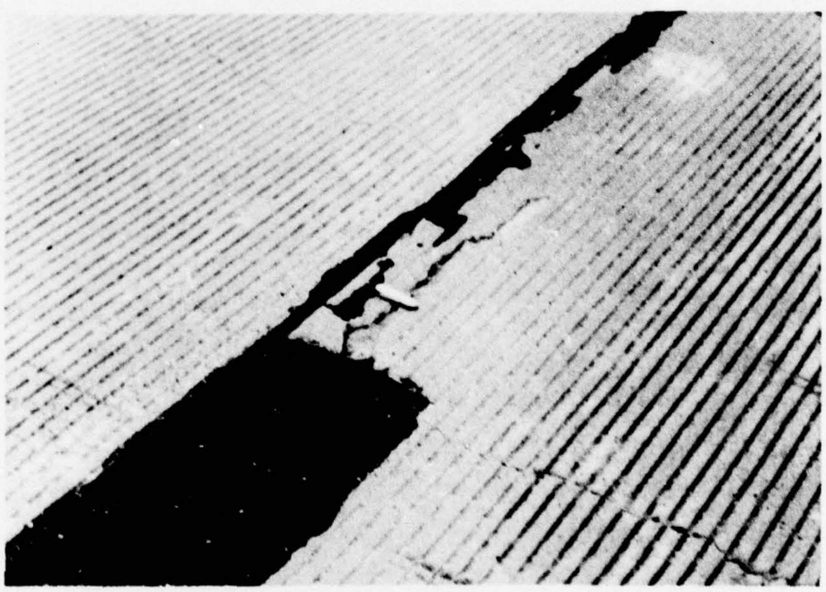
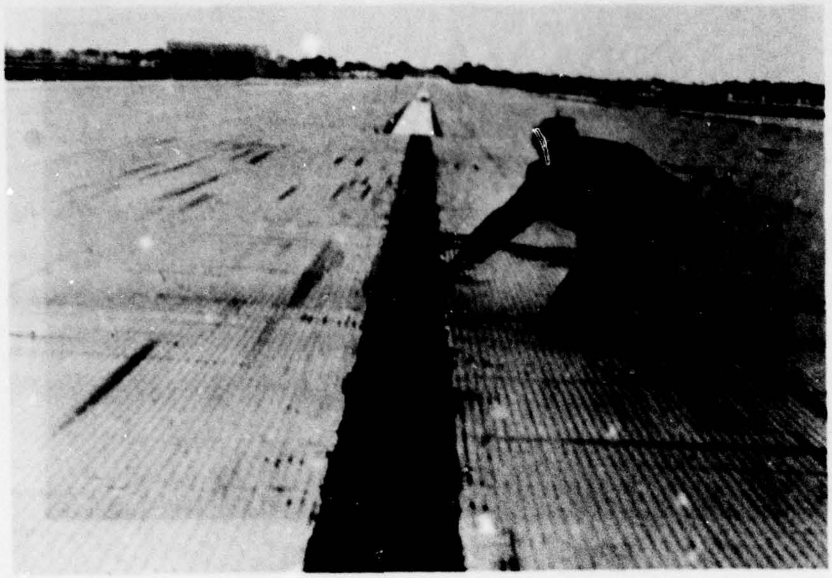


Figure A-11 - Extended Patch of Longitudinal Joint Distress With Further Distress Indicated at the End of Repair. (Midway Airport)



Figuare A-12 - Extended Longitudinal Joint Repair at Midway Airport.

Stapleton International Airport-Denver

Inspection of the runway and taxiway pavements showed significant distress in the longitudinal keyed joints especially in the pavement sections 13 to 16 years in service. While much of the actual distress occurred along the longidutinal joint, there was strong evidence to suggest that the distress was actually caused by deterioration of the concrete in general.

Figure A-13, A-14, and A-15 show some distress typical of that occurring along the longitudinal joints, especially in the taxiway A and to some extent in runway 1L-19R. This distress was often accompanied by surface cracking similar to that shown in Figures A-16 and A-17. While the cracking shown in Figures A-16 and A-17 resemble "D" cracking, in some ways it is also quite different from "D" cracking. One particular way it is significantly different from typical "D" cracking distress is that the surface crack patterns often tend to be distributed more or less uniformly throughout the slab. This is in contrast to typical "D" cracking distress which normally initiates near a crack or joint and propagates toward the center of the slab. The photographs in Figure A-18 taken by the Stapleton Airport staff show the progression of the distress patterns in these pavements. The photographs cover a period from December 1973 to May 1974.

To obtain a better understanding of the distress mechanisms associated with this distress, it was requested that the Stapleton staff take 3 cores from a slab showing the type distress observed, one core should be taken adjacent to or across a joint, one core approximately one foot from the joint, and the other core from the interior of the slab. Figure A-19

shows photos of the three cores obtained. Note that none of the core recoveries were full depth, even the core from the interior of the slab. This is somewhat at variance with the patterns observed with classical "D" cracking deterioration of concrete slabs. Furthermore, while "D" cracking is usually associated with breakdown of the larger aggregate particles, the deterioration observed in the cores appeared to be in the bond between the cement paste and the larger aggregate particles. A white powdery substance at the failed aggregate-paste interfaces was observed, suggesting that the deterioration may be due to aggregate-cement reactions rather than to deterioration of the aggregate per se.

Deterioration of the concrete was a significant factor in the keyway distress at Stapleton Airport, but there was also some of the more classical keyway distress without apparent concrete deterioration shown in the other areas (Figure A-14).

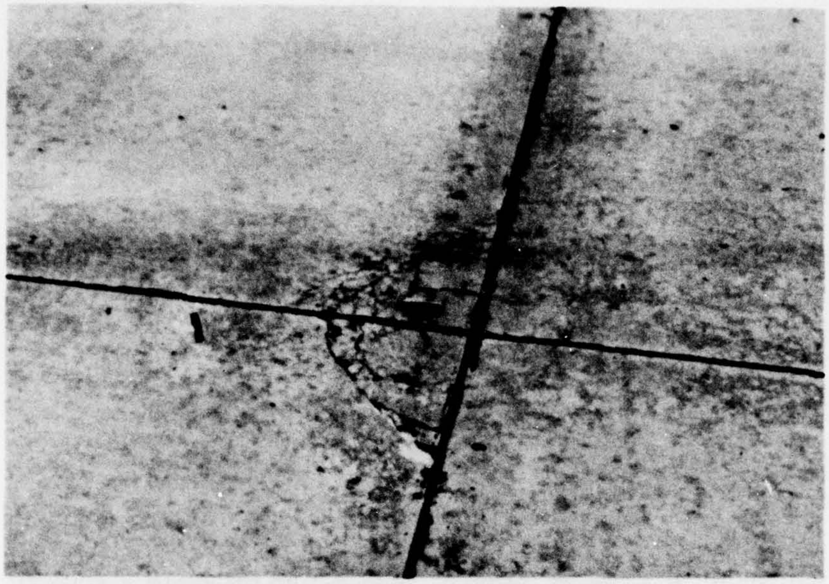


Figure A-13. Longitudinal Joint Distress Near a Transverse Joint, Stapleton Airport.

Figure A-14. Longitudinal Joint Distress Between Transverse Joints, Stapleton Airport.

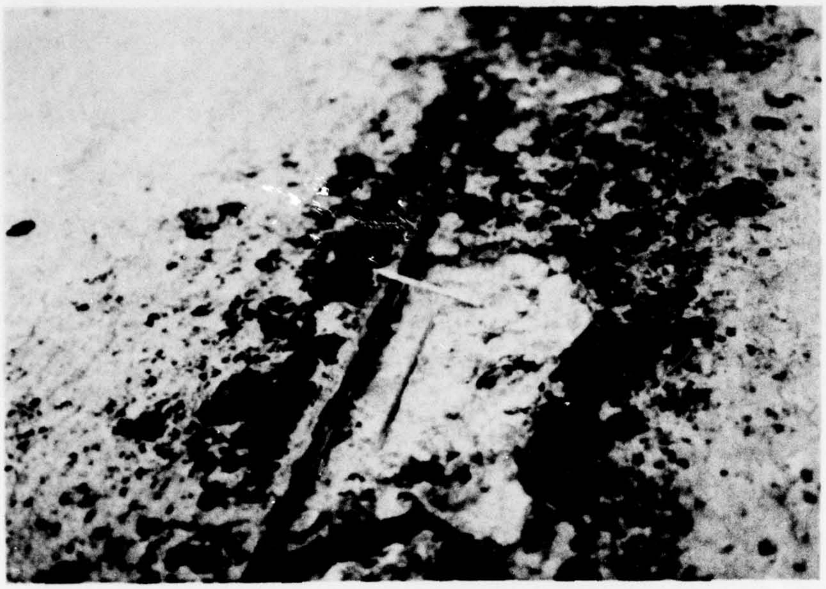


Figure A-15. Longitudinal Joint Distress Caused by Concrete Deterioration, Stapleton Airport.

Figure A-16. Surface Cracking near Intersection of Longitudinal and Transverse Joints, Stapleton Airport.

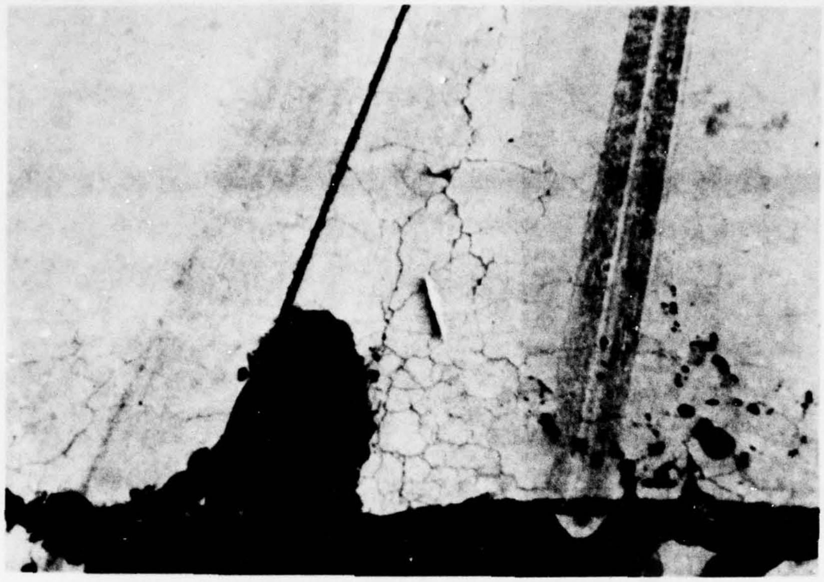


Figure A-17. Surface Cracking and Joint Distress, Stapleton Airport.

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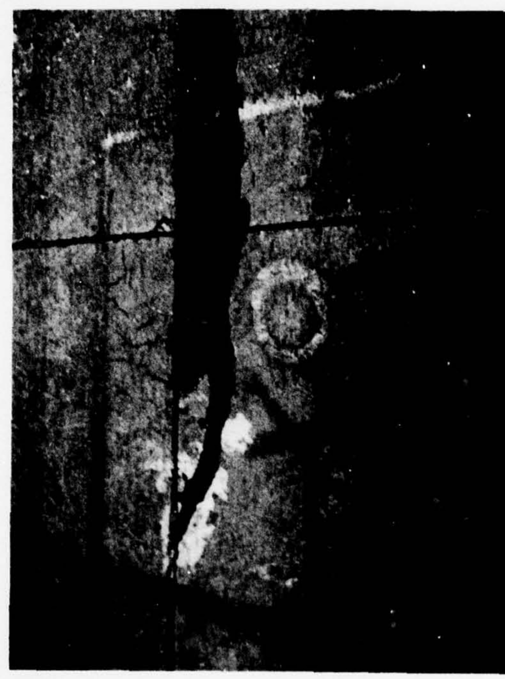

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Figure A-18. Photographs Showing the Progressive Deterioration of Concrete Near a Joint-Stapleton Airport.

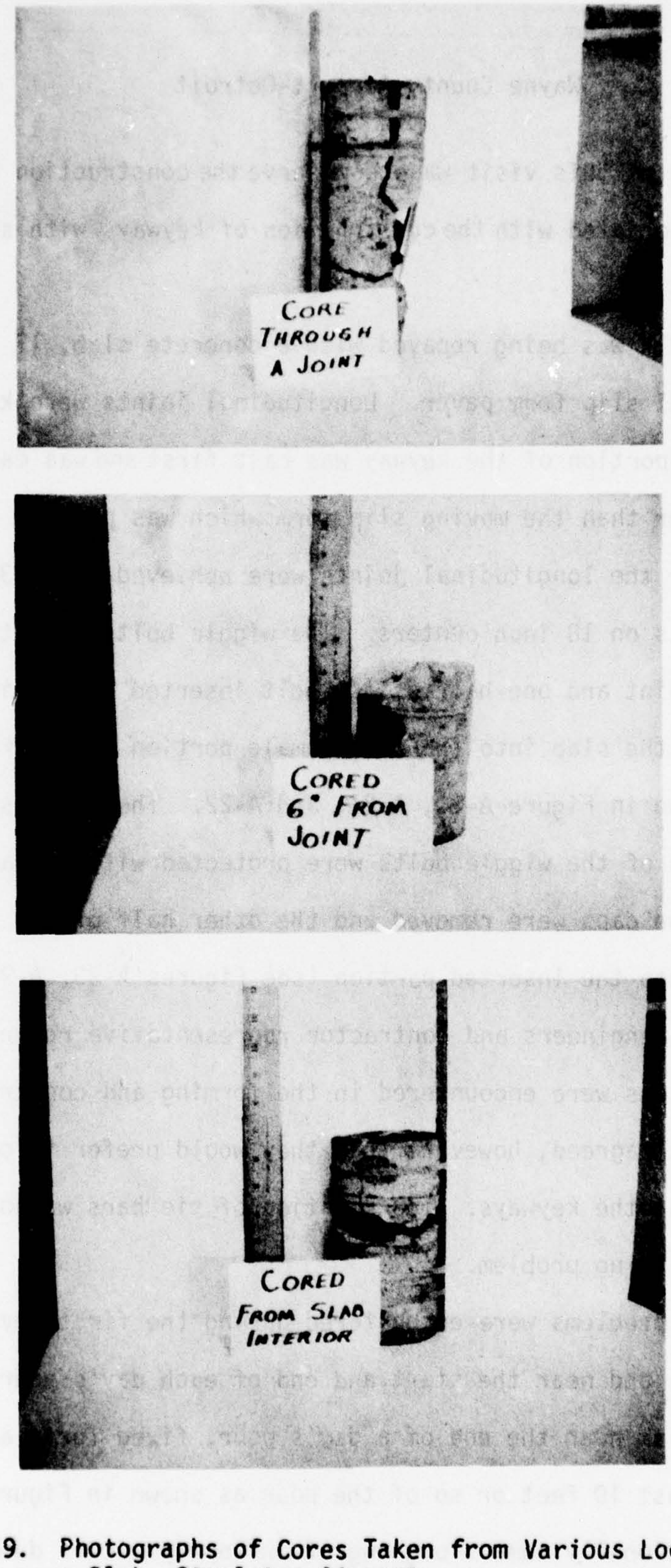


Figure A-19. Photographs of Cores Taken from Various Locations in a Slab, Stapleton Airport.

Wayne County Airport-Detroit

The purpose of this visit was to observe the construction procedures and problems associated with the construction of keyways with slip formed pavements.

Runway 3L-21R was being repaved with a concrete slab, 17 inches thick using a CMI slip form paver. Longitudinal joints were keyed and tied. The male portion of the keyway was cast first and was cast without any support other than the moving slip form which was part of the finish paver. Ties for the longitudinal joints were achieved using 3/4 inch round wiggle bolts on 18 inch centers. The wiggle bolts were threaded near their midpoint and one-half of the bolt inserted hydraulically near the midpoint of the slab into the formed male portion of the keyway concrete as shown in Figure A-20, A-21, and A-22. The threads in the inserted portion of the wiggle bolts were protected with a plastic cap. Subsequently, the caps were removed and the other half of the wiggle bolts screwed into the inserted portion (see Figures A-23, A-24, and A-25).

The airport engineers and contractor representative reported that no particular problems were encountered in the forming and construction of the keyways. All agreed, however, that they would prefer to construct pavements without the keyways. Installation of tie bars without the keyways was reported to be no problem.

Some minor problems were encountered during the first several days of construction, and near the start and end of each day's pour. To resolve the problem near the end of a day's pour, fixed forms are used for the first and last 10 feet or so of the pour as shown in Figures A-26 and A-27. Occasionally some sloughing of the keyway occurs due to the installation of the tie bolts. This is particularly true if the slump

is too high or if it rains during finishing or just after. When the keyway does slough off as shown in Figure A-29, it is replaced by dowels 1 1/2 inches in diameter by 20 inches long, on 18 inch centers. These are usually drilled into the hardened concrete and grouted into place. Figure A-29 shows two such dowels in place in areas where the keyways had sloughed for a short distance.

Further discussions with the airport engineers indicated no significant keyway failures with the thicker pavement designs. Some isolated keyway failures apparently had occurred in one of the cargo areas off 21R.

Both contractors and the airport engineers agreed that the construction of keyways in slip formed concrete pavments could be done but preferred not to do so. Installation of ties are no problem with butt joints in plastic concrete, i.e., if there are no keyways. Installation of dowels in plastic concrete caused more problems than installation of tie bars because of the larger bar diameter. A more effective but also more expensive way of installing dowels is to drill into the hardened concrete and grout the dowels into the concrete. Quick setting epoxy grouts are normally used for this purpose.

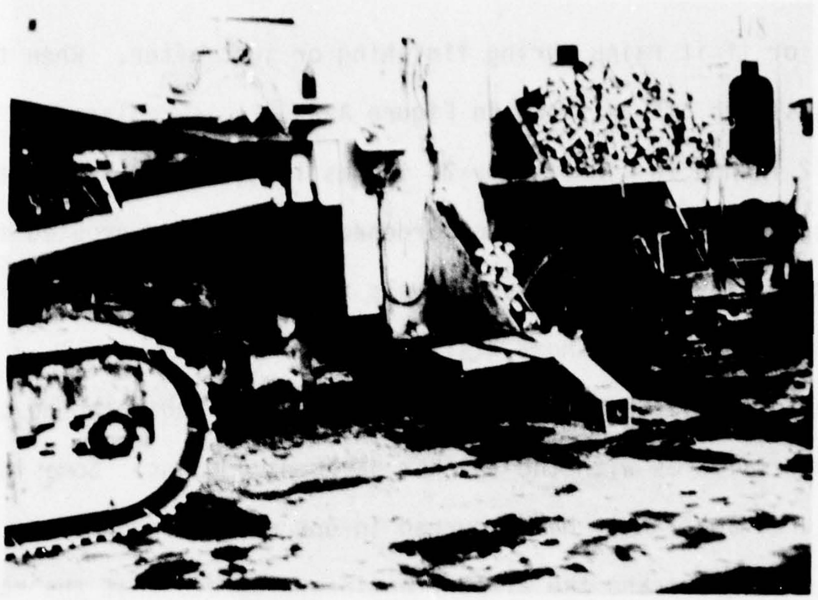


Figure A-20. View of Paver Showing a Supply of Wiggle Bolt Ties and the Hydrualic Ram Used in Installation into the Plastic Concrete, Detroit.

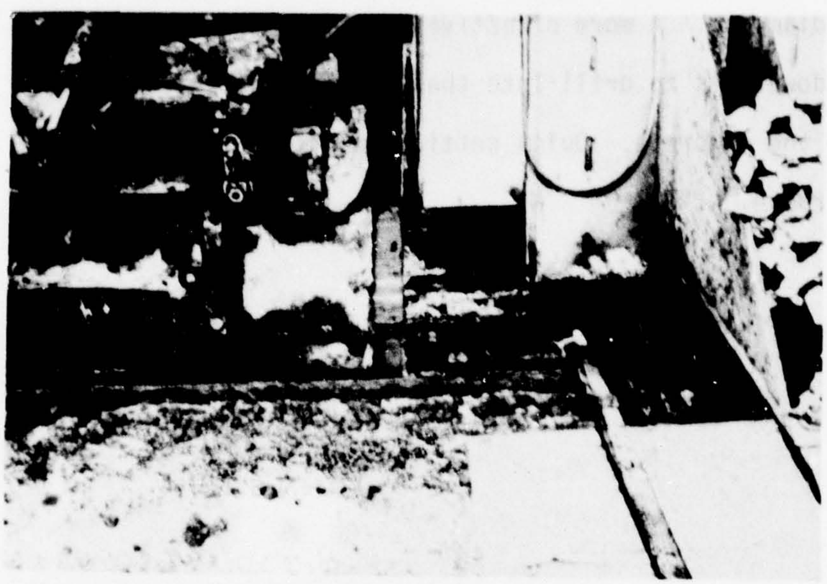


Figure A-21. Closer View of Wiggle Bolt Installation Equipment, Detroit.

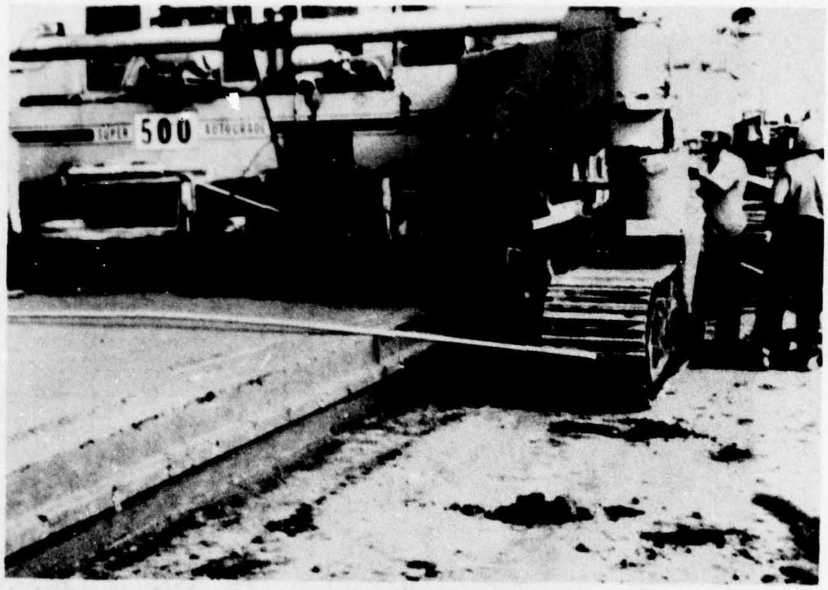


Figure A-22. View of Finished Keyway with First-Half of the Wiggle Bolts in Place, Detroit.

Figure A-23. View of Keyway with Second-Half of Wiggle Bolt in Place, Detroit.

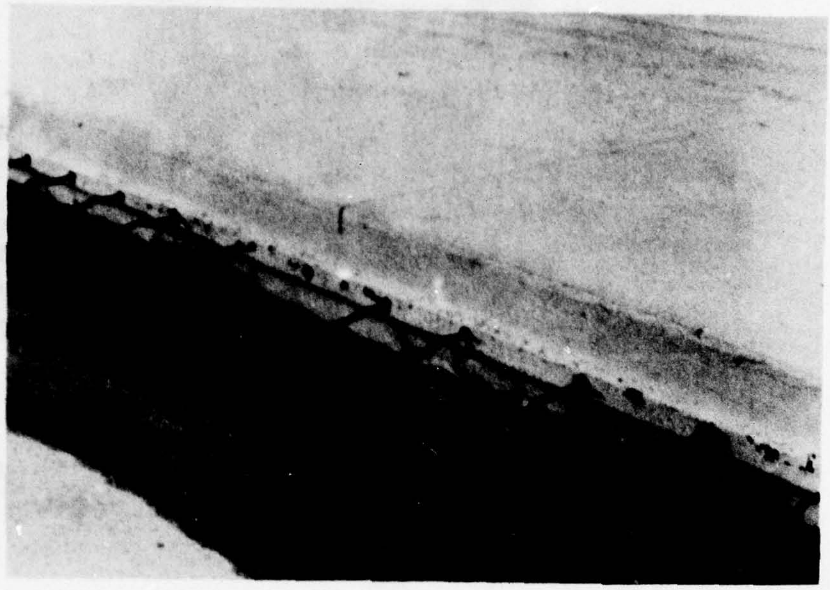


Figure A-24. Further View of Keyway and Wiggle Bolts in Place, Detroit.

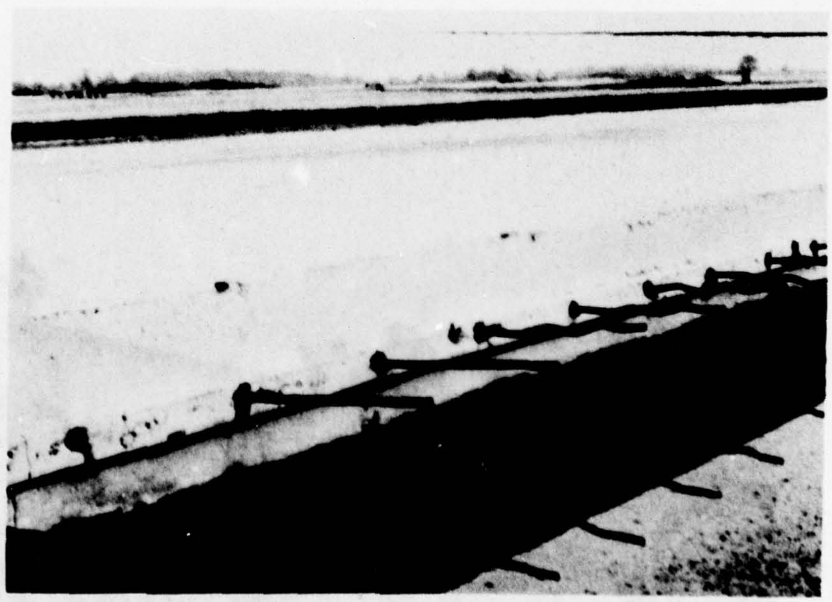


Figure A-25. View of Keyway and Wiggle Bolts in Place with Missing Bolt, Detroit.

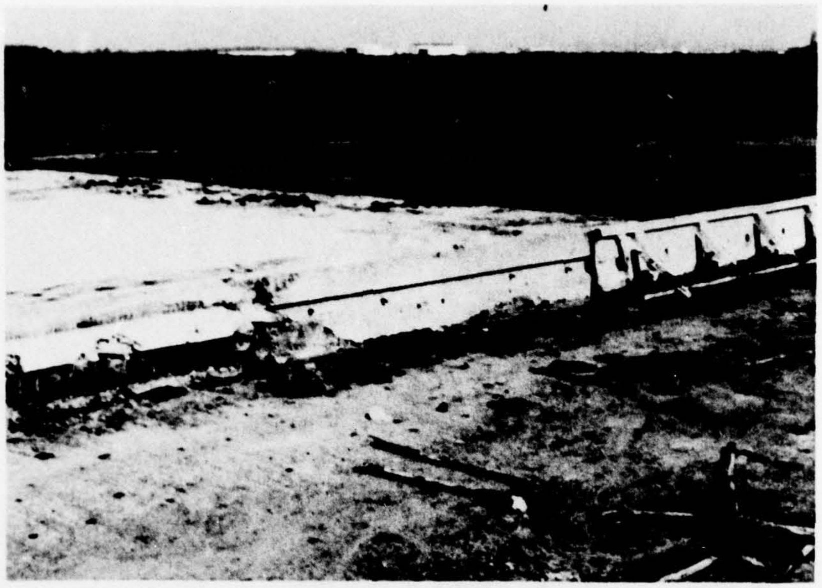


Figure A-26. Fixed Forms Used at the Beginning and End of Each Pour, Detroit.

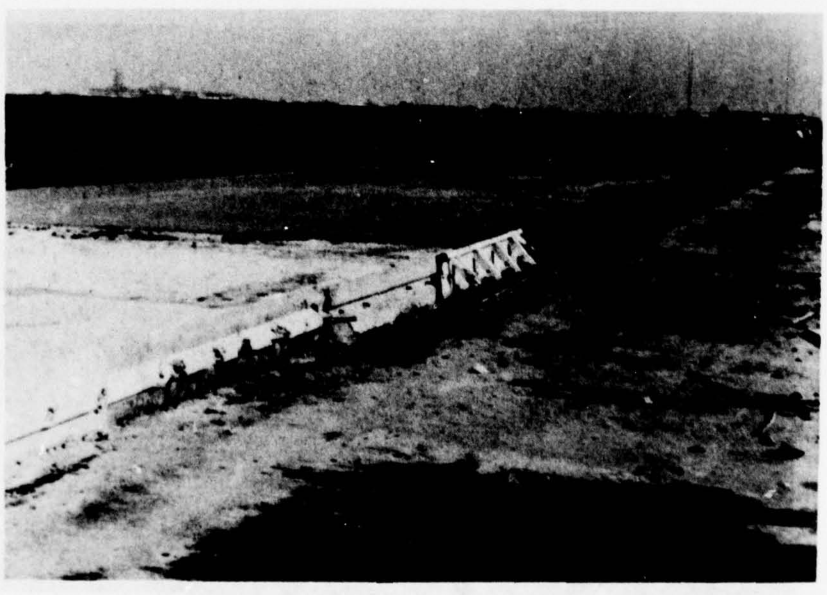


Figure A-27. Fixed Forms Used at the Beginning and End of Each Pour, Detroit.

Figure A-28. Sloughed Keyway, Detroit.

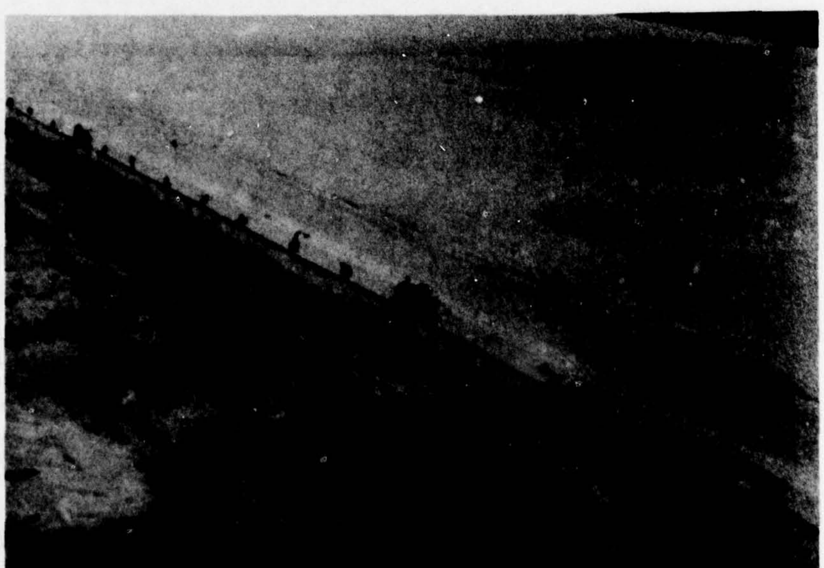


Figure A-29. Dowels Replacing Sloughed Keyway, Detroit.

Kansas City International

A brief visit was made to the KCI Airport to observe the types of distress in the airport pavements. Three types of distress as shown in Figures A-30, A-31, and A-32 predominated on the pavement.

A major type distress throughout the airport pavements is the longitudinal cracking as shown in Figure A-30. This occurred most often in the outside paving lanes, away from the major traffic patterns. This distress was probably due to temperature variations through the slab.

The cracking shown in Figure A-31 was generally observed in the heavy traffic lanes. It is probably due to inadequate slab thickness for the loads imposed and poor drainage conditions as illustrated in Figure A-33.

A major problem throughout the airport is "D" cracking in the concrete as indicated in Figure A-32. In some instances this "D" cracking has progressed through the entire slab as shown in the Figure.

No keyway failures were observed on the KCI Airport pavements.

Figure A-30 - Longitudinal Cracking, Kansas City.

Figure A-31 - General Pavement Failure - Kansas City

Figure A-32 - Failure of Concrete due "D" Cracking. Note the Deterioration has Destroyed the Concrete Completely Through the Slab, Kansas City Airport (Wall).

Figure A-33 - Surface Drainage Caused by Non Uniform Settlement of Pavement Slabs.

Wier Cook Airport - Indianapolis

Keyway distress at Wier Cook was very severe. Figures A-34, A-35 and A-36 show the extent of the longitudinal joint keyway failures at various stages of development and repair. Within the heavily trafficked portions of the runways, well over 50 percent of the keyways in the longitudinal joints had failed (Figure A-37). There are several possible reasons for this.

Pavements at Wier Cook were 11 inches thick and significantly under-designed for the traffic currently using the airport. There was also significant "D" cracking of the concrete along both the longitudinal and transverse joints (Figures A-38, A-39 and A-40). It is not known for certain whether there is a correlation between deterioration of the concrete as evidenced by the "D" cracking and keyways failures, but it appears that the keyway failure is more severe in areas where there is also significant "D" cracking.

A pavement test and evaluation report, prepared by Isbill Associates, Inc., Consulting Engineers dated 1975 indicated that load transfer, as indicated by the dynaflect test results, at almost all joints, was very poor.

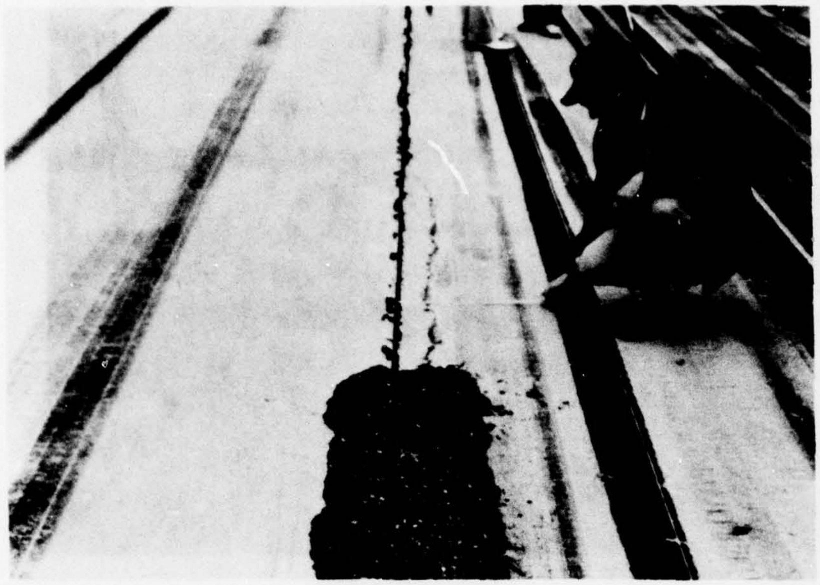


Figure A-34 - Longitudinal Keyed Joint Distress, Indianapolis.

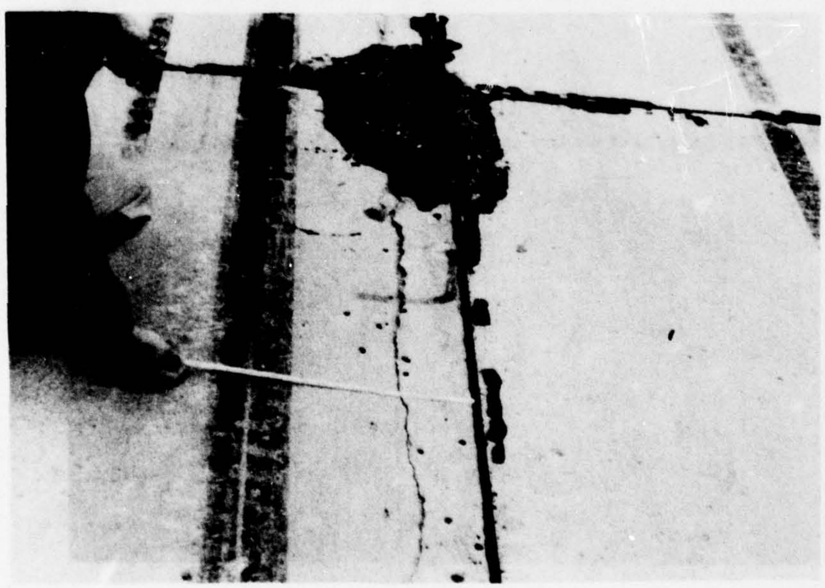


Figure A-35 - Longitudinal Keyed Joint Distress, Indianapolis.

Figure A-36 - Longitudinal Keyed Joint Distress and Repair, Indianapolis.

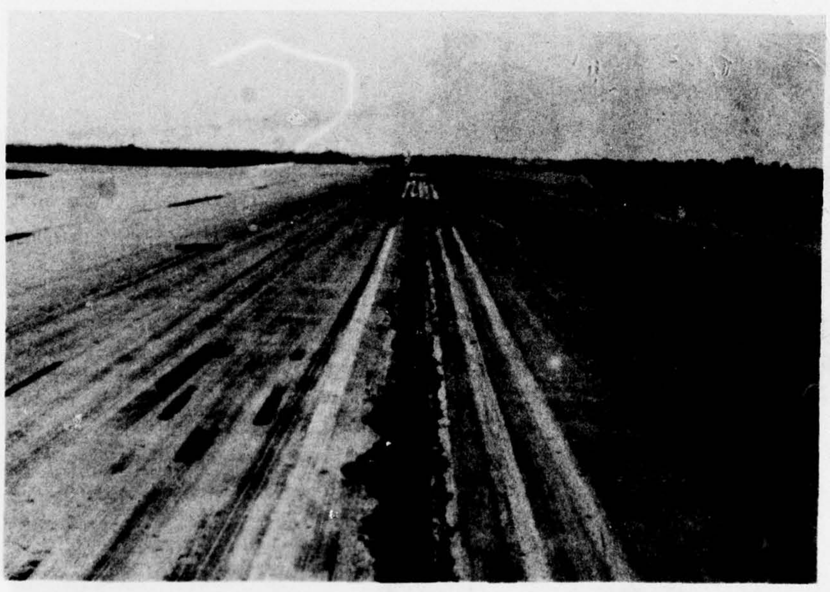


Figure A-37 - Longitudinal Keyed Joint Distress Repair Showing the Extent of the Distress, Indianapolis.

Figure A-38 - Concrete Deterioration, Indianapolis.

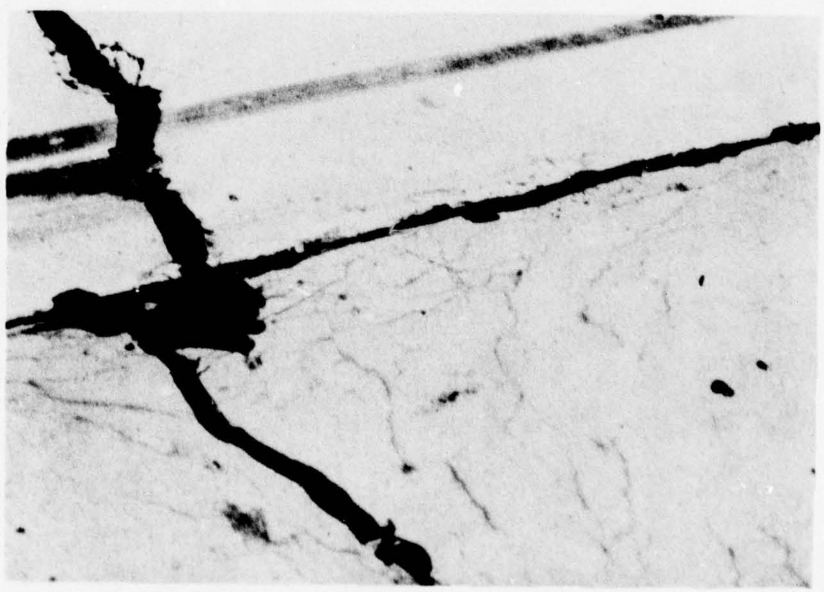


Figure A-39 - "D" Cracking Distress Indianapolis.

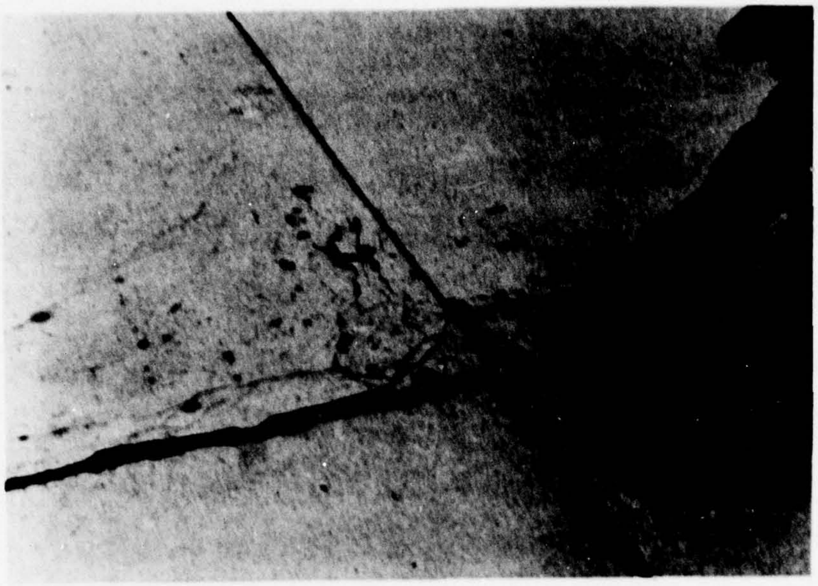


Figure A-40 - "D" Cracking Distress, Indianapolis.

Greater Pittsburgh International Airport

No inspection was made of actual pavement conditions during this visit. A meeting was held between the project staff and representatives of the consulting firm of Richardson, Gordon and Associates, which serves as engineers for the airport. In this meeting the construction problems associated with the construction of the longitudinal joint were discussed as were likely alternatives for a solution to the problems encountered. Most of the discussion was relative to problems encountered in the construction of the extension on runway 14-32.

The original design for this runway called for construction of the 16 inch slab in two lifts with the reinforcing steel placed on top of the bottom lift, and with the female side of the keyway formed first, using a metal insert to retain the keyway shape. The contractor proposed instead to pour the slab in one lift, placing the reinforcing steel mesh with depressors and forming the male side of the keyway first without benefit of support. To determine if this construction procedure was acceptable, the engineer proposed the following conditions and criteria:

1. A demonstration pour of 1,000 lineal lane feet of pavement with male key placed prior to approval.

2. The first 800 lineal feet for adjustment purposes only.

3. The last 200 lineal feet as the basis for approval or rejection. Approval requires 95% retention (190 L.F.) of the full section of the male key as formed, and satisfactory consolidation under the male key when the adjacent slab is poured. The Contractor must provide three 4" cores of the key to demonstrate satisfactory consolidation under the male key.

4. If the foregoing criteria are met, and the approval of the male key is granted, the contractor will reimburse the County a credit at the rate of $0.25 per lineal foot for the remaining slip formed, male keys placed.

5. If the foregoing criteria are not met, and the use of male keys is not approved, all longitudinal Type E joints will be either female keys, as specified, or changed to a Type D joint, using uncoated No. 10 deformed bars in lieu of dowels. No additional compensation will be allowed for either joint.

6. Remedial work shall be required in the demonstration area, and also for subsequent male keys if approved, whenever the full section of the male key is not retained over a length of 6 inches or greater. No. 10 deformed bars will be inserted at the center of the slab for load transfer at a spacing not to exceed 12". These bars shall be 20" long with 10" inserted in the plastic concrete.

7. Consolidation under the hook bolt and under the male key will be achieved by the Contractor by maintaining a continuous surcharge of concrete over the end of the spreader and by vibration as required by the director.

Using the conditions and criteria outlined above, the contractor tried three times to form the proposed male-female keyway system and failed each time. The final construction employed the Type "D" joint with uncoated No. 10 deformed bars in lieu of dowels. No particular problems were encountered in hydraulically inserting the No. 10 deformed tie bars into the plastic concrete.

As part of this discussion it was brought out that a slab of runway 10L-28R has been completed, but has not been opened to traffic, and probably will not be opened for several years. This section would make an ideal section for testing the relative efficiency of doweled longitudinal joints for load transfer. Also, taxiway "J" with 13 inches of concrete pavement, 7 inches of cement treated subbase, 24 inches of granular subbase and doweled longitudinal joints has had little traffic and would make an excellent test section for evaluating joint load transfer efficiency of doweled joints and for validation of models developed for the analysis of these joints.

A general discussion of the joint construction problems with the engineers brought out the fact that the butt joints without ties, dowels or keyways are easiest to construct, and therefore cheapest; that drilled dowel butt joints are probably the best but also the most expensive. Tie bars hydraulically inserted into the plastic concrete are effective and are relatively easy to install if there are no keyways to content with, particularly if the tie bars are small (1 inch diameter or less).

(SEA-TAC) International Airport - Seattle-Tacoma

A visit was made to the SEA-TAC Airport to observe some reported joint distress and to determine the performance of keyed longitudinal joints in thick pavement sections. The pavements in runway 16R-34L at SEA-TAC are thickened edge pavements with a cross section described by Brandly (1) as follows:

> Runway 16R-34L is 9.425 feet long and is the new runway which was constructed within the past 2 years. The basic pavement section consists of 14 inches of Portland Cement concrete over 19 inches of crushed aggregate base course. The subgrade soils consisted of compact silty sands and gravels. The PCC pavement was placed using a slip form paver in 37 1/2 foot wide lanes. Each longitudinal construction joint was thickened to 18 inches and was keyed. A few of the longitudinal joints were tied or dowelled, but most of these joints had no steel for load transfer. A longitudinal joint was saw cut between the longitudinal construction joints. Some of these joints were tied or dowelled, but most of them had no steel for load transfer. Transverse joints were saw cut on 20 foot centers. Most of these joints were neither tied or dowelled.

At the time of this visit, no evidence of longitudinal joint distress was observed. Some distress along the transverse joints, probably due to timing of the sawing operation, was observed, but these had been repaired and are performing well.

A report by Brandly (1) was obtained on loan from the SEA-TAC Airport authority which includes a discussion of performance of longitudinal joints as a function of the type of load transfer. This has been discussed in the final report for this study.

International Airport-Shreveport

The visit was made to Shreveport, LA to inspect the longitudinal joint and some other construction details, and to discuss the problems associated with construction of the keyway in the longitudinal joints with Mr. Ed Jolly. Mr. Jolly is a private consultant acting in the capacity as Engineer for the airport.

According to the airport engineer and designer, portland cement concrete overlays nine and one half and twelve and one half inches thick were designed with keyed longitudinal joints at 25' centers. Deformed bar ties were specified at 30 inch centers.

During construction it was attempted to form the male portion of the keyway first and to insert bent tiebars into the plastic concrete. No metal inserts or other supports were specified for the keyways. Insertion of the tiebars into the keyways of the plastic concrete caused the keyways to slough. These problems were sufficiently severe that the keyway was eliminated in the construction and straight tied butt joints used for most of the project. To date there are no apparent problems with these joints.

An interesting problem was observed along a centerline joint in a portion of one of the PCC overlays. The centerline joint was placed as a continuous pour across the existing centerline joint with deformed tiebars at 30 inch centers. The longitudinal centerline joint was sawed directly over the existing center line joint. For a substantial portion of the overlay (2000+ ft) a continuous crack developed parallel to and generally within six to eight inches of the sawed centerline joint. Occassionally this crack meandered as far as twelve inches from the sawed

joint, but generally it was much closer as shown in Figures A-41 and A-42.

The exact cause for the centerline joint is unknown, but it is hypothesized that the crack is a reflection crack from the centerline joint in the existing pavement. Because the crack did not follow the sawed joint as anticipated it is speculated that most of the cracking occurred before the joint was sawed. Temperature changes, particularly cooling, of the supporting slab would cause the existing slab joint to open thus initiating a crack near the bottom of the overlay. If this movement occurred prior to sawing the joint, it could be expected that the surface crack might meander somewhat but would remain fairly near the joint in the existing slab. This is exactly the pattern which developed.

At the time of the inspection the joint and crack were still in good condition. In some locations, however, loose pieces of concrete developed between the crack and the sawed joint. These were being retained in position by the hot poured joint filler which bonded to the spalled pieces of concrete. This particular joint would likely perform better if the longitudinal saw cut along the centerline had not been made leaving only the tightly closed crack meandering slightly around the pavement centerline.

Another interesting condition was observed in an area where a concrete slab 12 inches thick was placed over an "econocręte" subbase four inches thick. The econocrete had cracked transversely with approximately 37-1/2 foot spacing between transverse cracks prior to placing the concrete slab. Despite the 15 foot joint spacing in the concrete slab, many of the transverse cracks in the econocrete subbase reflected through the concrete slab as shown in Figures A-43 and A-44.

Considering the relative strength of a cured concrete slab 12 inches thick and the strength of a four inch layer of econocrete, it seems unreasonable to consider that the econocrete subbase would cause a cured slab three times thicker to crack reflectively. This is particularly puzzling as some of the reflective cracking occurred near a sawed dummy joint in the slab. These factors can be reconciled only if it is assumed that the basic crack formation in the pavement occurred before the concrete had attained significant strength, and probably before the dummy joints were sawed. This condition could occur if the freshly placed concrete caused a significant cooling of the econocrete subbase thus causing the cracks to open, initiating a crack in the bottom of the concrete while it was setting up. The only problem with this hypothesis is that the weather at the time of placement of the concrete was not particularly warm (max. daytime temperature about 80°F).

The distress described above is a factor to be considered in future designs using econocrete subbase and concrete slabs. A possible way to resolve this problem is to place a slip layer or bond breaker between the econocrete and the slab. Either single or double plastic sheets or a thin layer of unbound sand or gravel could be tried for this application.

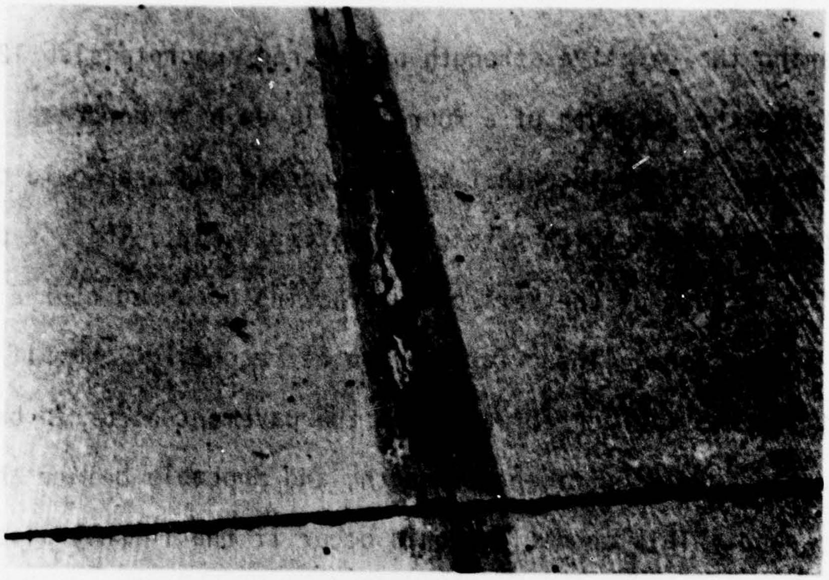


Figure A-41 - Cracking Distress Along Centerline Joint in Concrete Overlay, Shreveport

Figure A-42 - Cracking Distress Along a Centerline Joint in Concrete Overlay, Shreveport.

Figure A-43 - Reflective Crack Through Concrete Slab on an Econocrete Subbase, Shreveport.

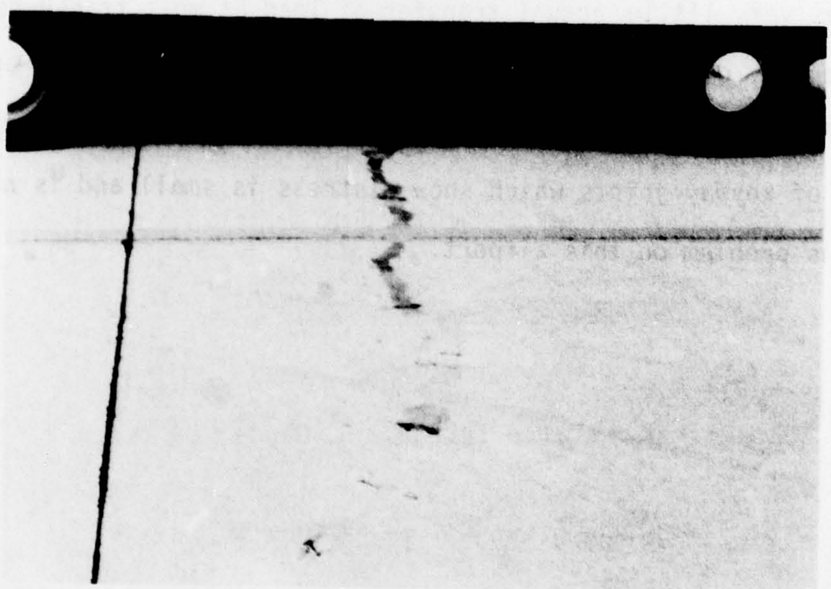


Figure A-44 - Reflective Crack Through Concrete Slab on an Econocrete Subbase, Shreveport. Crack was Repaired with Epoxy.

International Airport-Spokane

At the suggestion of the FAA Regional Pavement Engineer for the North-west Region, a visit was made to Spokane, Washington to observe the pavement conditions on the airport. The inspection revealed a number of potential problem areas on the airport pavements but only isolated cases of keyway failures, and these occurred mostly in the ramp areas.

Among the more serious problems observed are the general deterioration of some of the older concrete, and spalling along the transverse joints of the runways and taxiways. These distresses are of particular concern as the deteriorated concrete tends to spall leaving loose debris on the pavement which is subjected to ingestion by the jet engines. Also, spalling along the joints reduces the load transfer potential of the joints. A recent report by Edward A. Nurse, Engineering Consultant for the airport indicates very little actual transfer of load at most transverse joints.

The only evidence of keyway failure was observed in the apron pavement. Figures A-45 and A-46 show typical keyway failures observed. The percent of keyway joints which show distress is small and is not currently a serious problem on this airport.

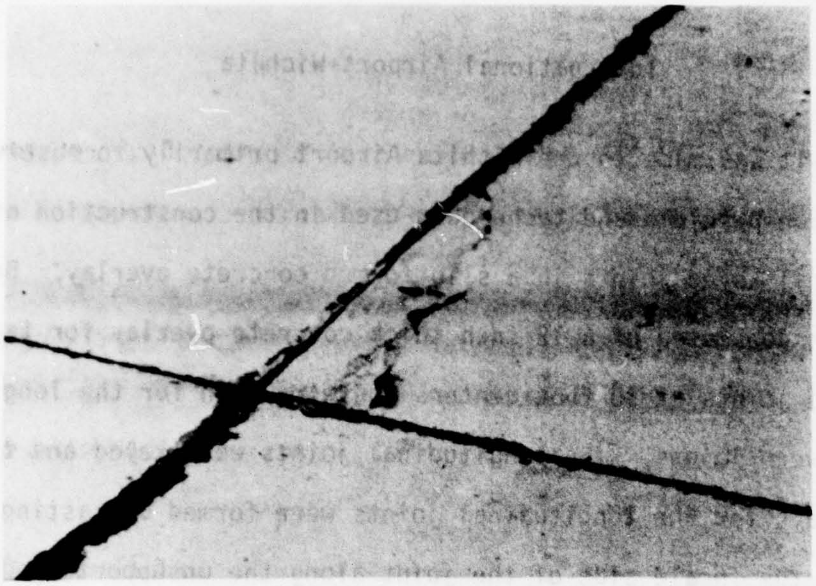


Figure A-45 - Typical Keyway Distress in Pavements in Apron Area, Spokane.

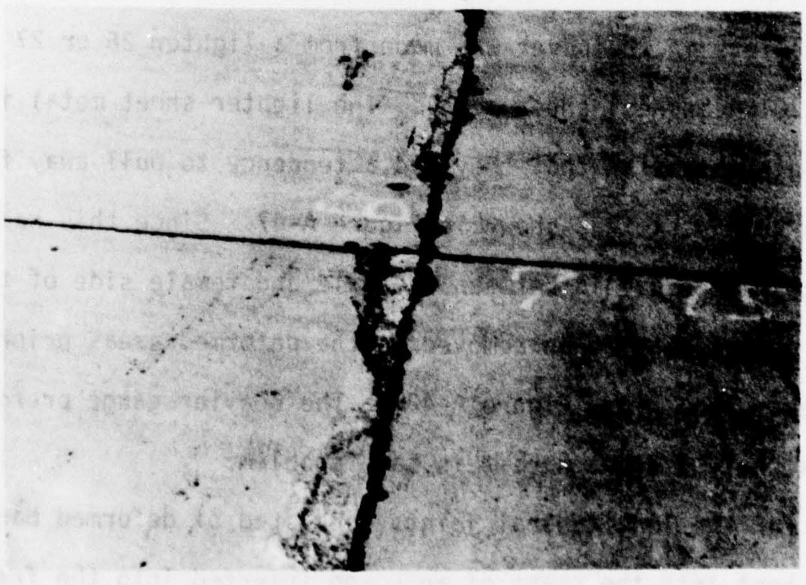


Figure A-46 - Typical Keyway Distress in Pavements in Apron Area, Spokane.

International Airport-Wichita

A visit was made to the Wichita Airport primarily to observe the construction problems and techniques used in the construction of the longitudinal keyed joints in a slip formed concrete overlay. Basically, the design consisted of a 12 inch thick concrete overlay for taxiway with transverse joints at 50 foot centers and wire mesh for the longitudinal steel between joints. The longitudinal joints were keyed and tied.

Keyways for the longitudinal joints were formed by casting a metal form into the female side of the joint along the unsupported edges of the pavement. Two types of metal forms were used, one a preformed insert supported on chairs at the appropriate height above the subbase surface, and the other type formed by a series of rollers in the paver. The preformed metal inserts were made of a relatively heavy gauge metal (24 gauge .0239 inches estimated) while the insert formed from a metal strip by rollers in the paver was made from a lighter 26 or 27 gauge (.0179 to .0164 inches) sheet metal. The lighter sheet metal inserts caused some problems in that they had a tendency to pull away from the top of the keyway slot as shown in Figure A-47. Since this had the potential to produce a void between the male and female side of the keyway the metal inserts had to be removed in the deformed areas prior to placing the adjacent slab. (See Figure A-48). The heavier gauge preformed inserts apparently did not have this same problem.

Ties for the longitudinal joints consisted of deformed bars. The bars were bent into the shape of an L and inserted into the fresh concrete through holes prepunched in the metal inserts. After the concrete had set the bars were straightened to form a tie across the keyed joint. The only problems encountered with this procedure is that if the bars are initially bent too sharply, they may fracture when straightened.

As a general rule, the construction of the keyway in the longitudinal joints did not present any significant problems. The keyways and pavement edges remained true to the formed grade and the final pavement edges appeared true and even.

One reason for the lack of problems in this job was likely the apparent quality control in the concrete mix operations. Both the concrete batch plant operation and the paving operations were under the same management (Standard Construction Co.) so there was no splitting of responsibilities. The slump of the concrete was monitored on a more or less continuing basis by observing the amps of electricity required to operate the drum motor. As the slump of the concrete increases, the power requirements for operating the drum decreases. Thus, by keeping track of the power requirements on the drum motor, the slump of the concrete can be controlled.

In addition to the construction operations a number of types of pavement distress was also observed. Figures A-49 and A-50 show several cases of longitudinal joint distress. Only Figure A-51 shows a true keyway failure pattern, while the remaining distress was more likely due to construction problems along the edge. "D" cracking as shown in Figure A-52 and A-53 is a problem throughout the airport on all pavements over 5 to 8 years of age. This "D" cracking deterioration has apparently not resulted in any significant increase in keyway failures as suggested for some other airports visited.

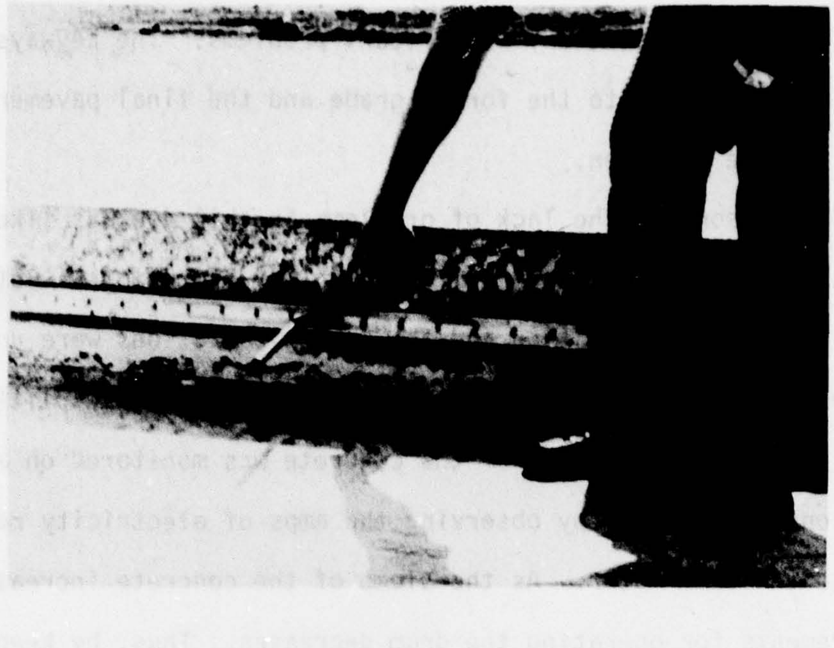


Figure A-47. Keyed Longitudinal Joint with Gap between Metal Insert and Top of Keyway, Wichita.

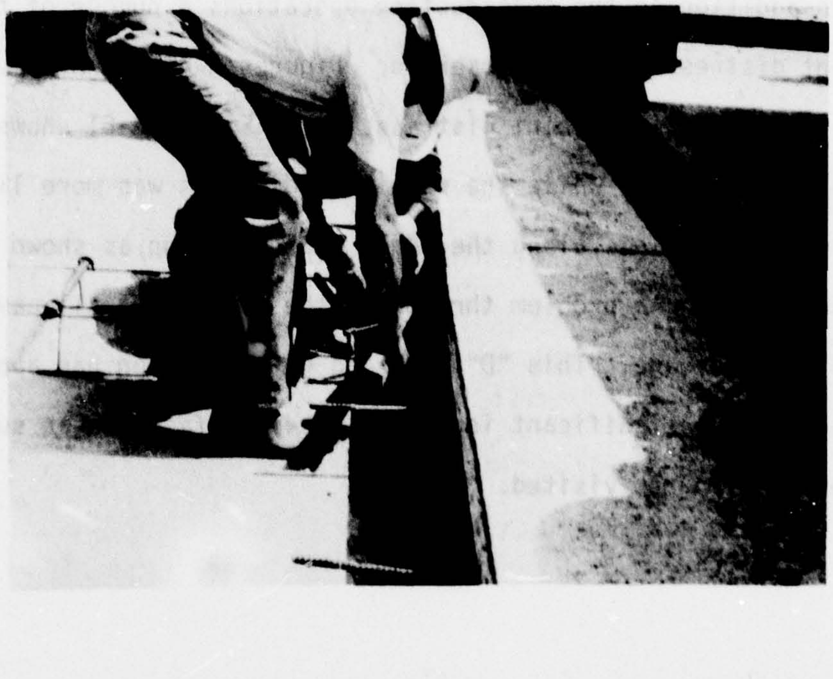


Figure A-48. Removal of Metal Keyway Insert in Areas where Gap Occurred, Wichita.

Figure A-49 - Longitudinal Joint Distress Repair, Wichita.

Figure A-50 - Corner Distress Patching, Wichita. Most Corner Distress Related to "D" Cracking.

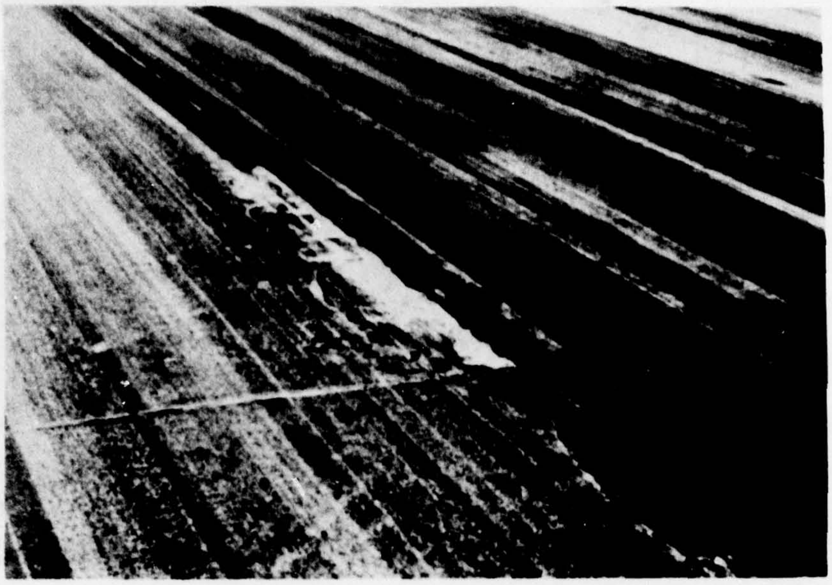


Figure A-51 - Longitudinal Distress of Keyed Joint, Wichita.

Figure A-52 - "D" Cracking Distress With Restraint Cracking, (Wichita).

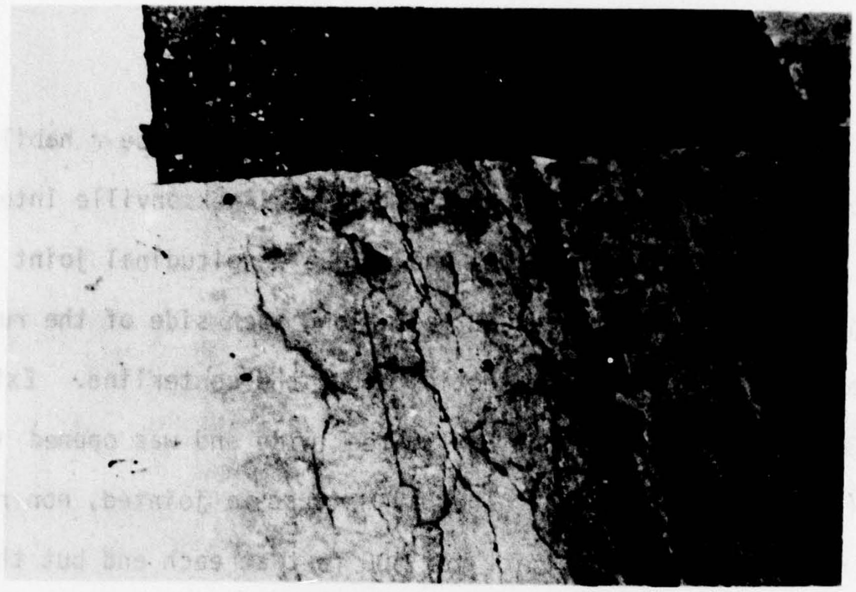


Figure A-53 - "D" Cracking Deterioration, Wichita.

International Airport-Jacksonville

The trip to Jacksonville, Florida was to observe the r habilitation program being undertaken on runway 13-31 at the Jacksonville International Airport. Of specific interest was the unique longitudinal joint details used to join the in-place outer 50 feet along each side of the runway with the replaced 50 foot wide keel section along the centerline. Existing runway 13-31 is 7700 feet long by 150 feet wide and was opened to traffic in 1967 (10 years of service). The pavement is a jointed, non-reinforced concrete pavement 13 inches thick for 500 feet at each end but the center 6700 feet is 11 inches thick. A non-stabilized granular subbase is under the entire length. The subgrade soil is variable and includes silty-sandy soils to compressible organic clayey soils. No subsurface drainage was provided.

A 50 foot width along the centerline for the 6700 feet of 11 inch pavement had deteriorated badly but the outside 50 feet was in good condition. The end 500 feet of 13 inch pavement was also in good condition except for some minor spalling and keyway failure along the longitudinal joints. Thus, it was decided to remove the center 50 x 6700 foot section and to replace it with a keel section of greater structural capacity.

The attached write-up taken from the American Concrete Paving Association Newsletter, Volume 13, NO. 2, February, 1977, describes the keel section being inserted into the runway. The longitudinal drains shown were intercepted with transverse drains every 250 feet. A porous, non-woven fabric was placed over the subgrade soil to prevent the soil from infiltrating the filter layer.

"One of the most innovative runway strengthening projects is presently under construction at Jacksonville, Florida, by ACPA member Claussen Paving Company, Augusta, Georgia. The project involves strengthening Runway 13-31 by removal of a 50-foot keel strip and recycling the removed concrete to provide aggregate for a drainage layer and econocrete subbase. A new pcc pavement tapering from 14 inches to approximately 20 inches will be built as shown in the accompanying drawing.

Pavement removal, excavation, crushing of the pcc pavement to produce aggregate, and placement of the drainage blanket are underway at the present time.

In order to prevent any future spalls, a full-depth cut was made to remove the male keyway from the 11-inch pavement to be left in place. Load transfer at this joint was provided by undercutting the existing slab and providing a "shelf" support as shown in the drawing.

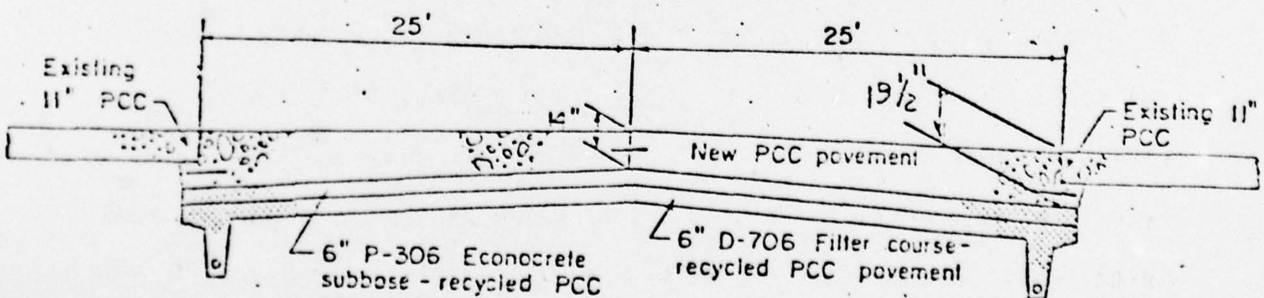
Other innovations include an econocrete subbase which will use recycled pcc pavement as aggregate. The plastic econocrete mix is particularly suited for use on this project since it is consolidated by internal vibration of slipform equipment or hand placing methods. The water table is close to the subgrade surface, and compaction of other types of bases to proper density could prove difficult.

Another innovation for airports is the use of a filter fabric between the subgrade and the filter course made of recycled pcc pavement.

For anyone planning to visit the project, the first week in March would be an appropriate time. Operations scheduled at that time invlude excavation, crushing of pcc to produce aggregate, placement of the Mirafi filter fabric and the recycled aggregate drainage layer, and slipforming of the econocrete subbase using recycled pcc aggregate.

Jay Dresser, Engineer with the Jacksonville Port Authority, has been instrumental in the development of this project since its inception. Mr. Dresser can provide information on the project and its scheduling if you plan a visit. His telephone number is (904) 633-5848."

Jacksonville International Airport
Runway Strengthening Using Recycled
PCC Pavement



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